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A WINTER ENGINEERING PARTY.

Those who have and those who have not engaged in surveying work in cold northern countries will, perhaps, be equally interested in the accompanying reproduction of a photograph of an engineering party engaged in running a preliminary railway line from Gun Flint Lake, on the United States and Canada boundary line, to the Highland Station of the Duluth & Iron Range R. R., a distance of 86 miles. The party consisted of Mr. E. T. Abbott, Chief Engineer, 12 white men, 10 Indians and 8 dogs. Mr. Abbott may be seen on the right of the view, holding the dog "Brecko." Mr. Abbott, to whom we are indebted for the picture, informs us that the work occupied about three months in the dead of winter, and he adds that it was "accomplished without accident, and, what is out of the ordinary, the cook stuck till the job was finished." The cooking was done for the 22 men on a special stove weighing only 80 lbs. The stove was only 22 ins. wide, as a greater width would have made trouble in getting it between trees on a toboggan. Mr. Abbott is now located at Thrall, Cal., where he fills the position of general manager of the Klamath Lake R. R., besides a variety of other positions.

A WINTER ENGINEERING PARTY ON A RAILWAY SURVEY.

THE NEW STEEL VIADUCT BETWEEN KANSAS CITY, MO., AND KANSAS CITY, KAN.

An important and interesting steel structure which has recently been completed is the new viaduct connecting the cities of Kansas City, Mo., and Kansas City, Kan. This viaduct is 8,400 ft. long, and crosses the Kaw River valley and bottom. The former of these cities, with a population of about 200,000, is located partly in the valley and upon the hills of the east side; the other city, with a population of about 80,000, is similarly located in and upon the west side. The extensive railway freight terminals are practically all located in the Kaw Valley, but on the Missouri side of the state line.

The difference in the elevation between the general level of the two cities and that of the bottom lands of the Kaw Valley is about 100 ft. The low lying district is practically a network of railway tracks, and owing to the heavy grades and the inconvenience of grade crossings the wagon traffic to and from the high land and the valley is greatly impeded. For a number of years the subject of a suitable connection between the various districts by means of free highway viaducts has been agitated, but the amount of money involved in the proper solution of the problem was so great that the cities have not seen fit to provide the necessary facilities. In 1904, Mr. Wm. J. Knepp, of Kansas City, Mo., conceived the plan of providing the necessary viaduct as a private enterprise, the income to be secured by collecting tolls. After due consideration of the problem, it was decided to provide also for two street railway tracks on the structure. A corporation, known as the Kansas City Viaduct & Terminal Co., was organized with a capital of $3,000,000, to build and operate the structure. The necessary franchises were obtained from the two municipalities, and the proposition was financed by New York and Boston bankers. Work was commenced in August, 1905, and the viaduct was opened to traffic on Jan. 29, 1907.

GENERAL DESCRIPTION.

A map of the viaduct, showing its connections and its relation to the street and street-railway systems in the two cities is shown in Fig. 1. It begins at Bluff St., Kansas City, Mo., as a continuation of Sixth St., which descends to Bluff St. on a grade of 3.5%. The viaduct has a grade of 1.5% for some distance, and then a

grade of 0.5% to about the middle of the structure; beyond this it begins to rise again with the same grade, which is followed by one of 1.5% extending to the end of the viaduct at the intersection of Fourth St. and Minnesota Ave. in Kansas City, Kan. There are three curves in the alinement; these are all of 3°, with lengths of 200 to 700 ft. The total length is 8,400 ft., and the distance between the terminal points by way of the Metropolitan Street Ry. Co.'s most direct line is 11,950 ft., or 3,550 ft. in favor of the viaduct. From Eighth and Wyandotte Sts. on the Missouri side to Fifth St. and Minnesota Ave. on the Kansas side, the distances are 11,850 ft. and 16,000 ft., or a saving of 4,150 ft. by the viaduct route. It will be seen that there are four lateral approach viaducts connecting with two different points in the lower portion of each city. These give an easy grade for traffic, avoiding the necessity of climbing the steep hills. On the Missouri side these approaches begin at the intersections of Ninth and Hickory Sts. and Eleventh and Mulberry Sts. These are 20 ft. and 24 ft. wide respectively, and are parallel for a part of the distance, uniting in a short piece of 30-ft. viaduct which joins the main viaduct.*

The viaduct provides for highway and street railway traffic, but the tracks are outside of the roadway, which is protected by railings on both sides. The viaduct company will lease the use of the tracks to the Metropolitan St. Ry. Co. and to the Kansas City & Western Ry., the latter being an electric interurban line.

The general character of the design of the viaduct is shown by the part longitudinal section, Fig. 2. For the most part it consists of 45-ft. plate girder spans; every eighth span, however, is a 30-ft. braced tower span. At the ends and at special points where railway tracks, etc., are crossed, there are spans ranging from 30 ft. to 107 ft. There are only three truss-spans, all of which are parallel-chord riveted lattice deck trusses. One of these, near the eastern end, is a skew span 147 ft. long and crosses the Kansas City Southern Ry. The other two are near the western end, and carry the viaduct over the Kaw River; each of these is 300 ft. 10 ins. c. to c. of bearings. The girder spans and the 147-ft. truss spans are carried by steel columns on masonry pedestals. The viaduct has a roadway (but no sidewalk), and two street railway tracks, outside of the roadway. Provision is made for widening the structure in the future to carry two additional tracks. From the east end to the first lateral approach viaduct the width of roadway is 38 ft. on account of the local traffic between the higher and lower portions of the city. Beyond that the width is 30 ft. The tracks are 12 ft. 6 ins. c. to c., with trolley poles set between them.

It may seem strange that there are no sidewalks for pedestrian travel, but we are advised that these were not provided on account of the long distance across the structure, which is 1¹/₆ miles, and too far for any considerable pedestrian travel. When the structure was first opened there were a good many sight-seers who walked over the structure; but since then it has been found that (as anticipated) the pedestrian travel is extremely light and not sufficient to justify an expenditure of something like $100,000 to provide a sidewalk. The width of the roadway is considered sufficient to provide ample room for pedestrians without danger from teams and automobiles, unless, of course, there should be a runaway or careless driving. Nevertheless, it would seem that a sidewalk would be a desirable feature, or that a curb or railing should be provided for the protection of pedestrians, especially as passing vehicles may crowd the latter to the rail. At the terminals there are simply small wooden toll-houses, just sufficient [illegible] toll-keepers' shelter. Suitable switches [illegible] vided for railway connections at both end[s].

FOUNDATIONS AND SUBSTRUCTU[RE].

The entire length of the main viaduct [is] located upon alluvial soil, and in order to [obtain] sufficiently stable foundations it was necessary to drive clusters of piles for the pedestals of the columns. For some of the columns near the ends, however, the ground was firm enough to carry pedestals with footings. This is shown on the longitudinal section in Fig. 2. Owing to the fact that the low water level was about 30 ft. below the level of the tops of the pedestals, it was decided to use concrete piling in order to save the large amount of concrete that would have been necessary to carry the pedestals to this depth. The original idea was to use Simplex piles, which are made in place. As described in our issue of March 10, 1904, a steel pipe is driven to the required depth, the lower end of this being provided with a cast-iron shoe which opens when the driving is stopped. Concrete is then deposited inside of the pipe as this latter is withdrawn from the ground. After further consideration it was decided not to use this system, owing to the class of materials through which the piles were to penetrate. It was therefore decided to mold concrete piles, reinforced with rods, and after giving them time to harden to drive them in much the same manner as ordinary timber piles. These piles are made octagonal in cross-section, having a short diameter of 14 ins. at the large end (with sides 8 ins. wide) and 10 ins. at the small end. They are 30 ft. in length, and each was reinforced by

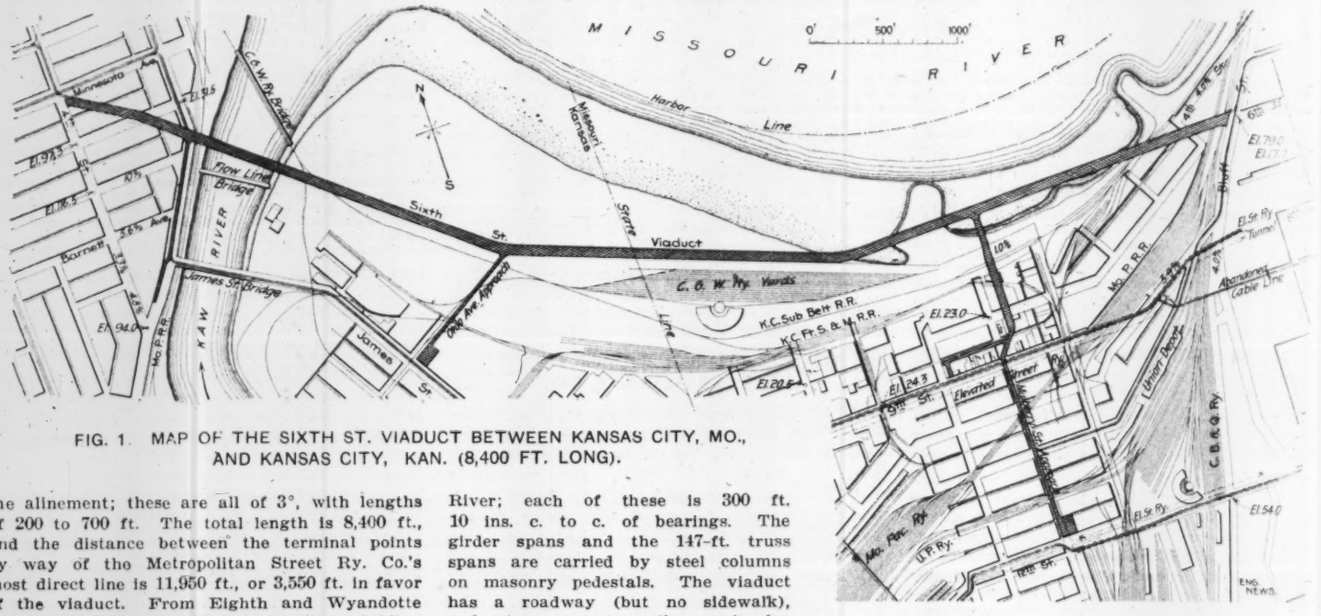


FIG. 1. MAP OF THE SIXTH ST. VIADUCT BETWEEN KANSAS CITY, MO., AND KANSAS CITY, KAN. (8,400 FT. LONG).

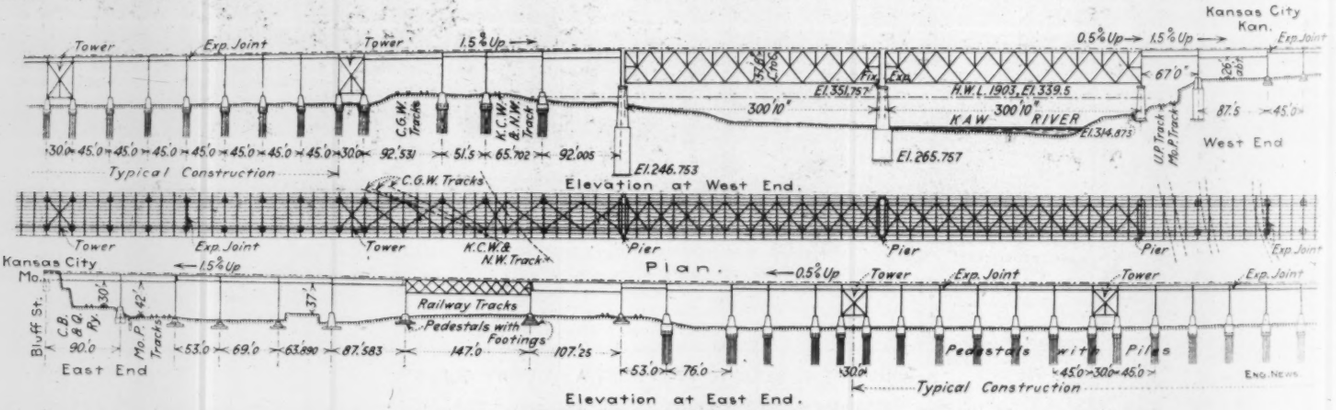


FIG. 2. PARTIAL ELEVATIONS OF VIADUCT.

*A description of the concrete approaches to the viaduct appeared in our issue of Jan. 31, 1907, p. 132.

four ¾-in. bars 15 to 20 ft. long, with rectangular hoops 4 ft. and 5 ft. apart, and a wrapping of wire netting. The molding of these piles was started late in August, 1905, and during the months of August, September and October very good results were obtained. The cold weather, however, later in the season so retarded the setting of the concrete that the further use of concrete piles that season was rendered impracticable. As the work would have been held up until the following spring, creosoted yellow-pine piles were substituted for the concrete piles, and the remainder of the work was completed in this manner. About 800 concrete piles and about 3,000 timber piles were driven.

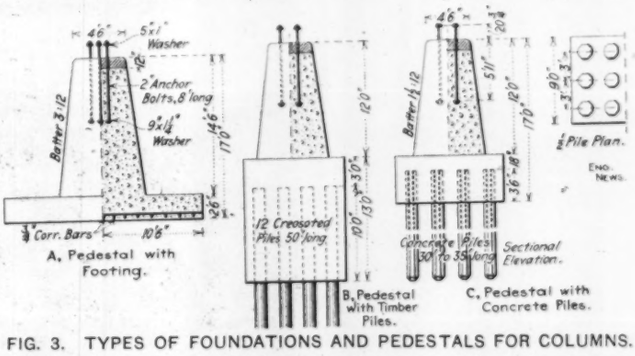


FIG. 3. TYPES OF FOUNDATIONS AND PEDESTALS FOR COLUMNS.

The three types of pedestal foundations are shown in Fig. 3. They are all of concrete, with a granitoid top finish, and are 4 ft. 6 ins. × 3 ft. 6 ins. at the top. Two anchor bolts are embedded in the concrete, and have washers at each end, as shown. At the right is one of the pedestals founded on firm ground; this has a 30-in. concrete footing 20 × 21 ft., reinforced at the bottom by two rows of corrugated steel bars. The middle pedestal shows the construction with wooden piles, and shows very clearly the depth of the concrete mass required to protect the piles above low-water level. The total depth of this concrete is 13 ft. below the pedestal. At the right is shown the construction with concrete piles; the concrete foundation block being here only 5 ft. deep, since the piles of course need no protection.

The concrete was made as a wet mixture, with proportions as noted below, the proportions being measured by volume. All reinforcement consists of Johnson corrugated steel bars.

Concrete.	Proportions.	Size of Stone.
Pedestals and foundations	1:3:5	2½ in. ring
Deposited in water	1:2:3	¾ in. screen
Granitoid top finish	1:2:3	*½ in. screen
Piles	1:3:5	1 in. ring
Piles concrete deposited in water	1:2:3	¾ in. screen

*Granite chips or Joplin flint.

PIERS FOR RIVER SPANS.—The piers for the two spans of the Kaw River bridge are all built upon bed rock. At the site of the west pier and west approach pier, rock was found above the water level. The middle and east end piers, however, were sunk to depths of 38.8 ft. and 56.8 ft., respectively, below the low water level. These two foundations were constructed by the pneumatic process. The construction of pier No. 2, with its caisson, is shown in Fig. 4, and Fig. 5 shows the sinking of the caisson for pier No. 1.

SUPERSTRUCTURE.

As already explained, the greater part of the viaduct consists of plate girder spans of 45 ft. and 30 ft., with special spans varying from 40 to 107 ft. In the typical construction there are bays of eight spans, including seven 45-ft. spans and one 30-ft. tower span. This is shown by Fig. 2. Each tower is braced longitudinally with rigid riveted members. Between each pair of towers is an expansion joint to allow for longitudinal movement; this is shown in Figs. 2 and 6. The cross-section, Fig. 7, shows the general arrangement, with the two widths of roadway already mentioned, and with the proposed future widening for two additional street railway tracks. The columns extend to the level of the tops of the girders, and to their webs are riveted the longitudinal girders and the cross girders. Cantilever girders carry one track and the outer end of the roadway. The structure is braced laterally by knee braces from the columns to the cross girders, these braces being placed at such a height as to give sufficient clearance for railway tracks beneath the structure. This is shown in Fig. 7. In addition to the cross girders over the columns, there are two intermediate cross girders in each 45-ft. span and one in each 30-ft. tower span. Upon these are placed the stringers for the roadway and the tracks.

All structural material is of medium open-hearth steel of from 60,000 lbs. to 70,000 lbs. tensile strength. The rivets are of soft steel, and are in general ⅞-in. diameter. The rivet holes are sub-punched to ⅛-in. smaller diameter and reamed to 1/16-in greater diameter than the rivets, except that in I-beam stringers and stringer bracing they were allowed to be punched full size. The ends of longitudinal girders and cross girders (or floor beams) and cantilever beams were faced and reamed in the field. The sheared edges of all plates were faced, with the exception of stay plates, lateral plates and lacing bars. All reaming for field connections was performed in the field after the work was erected.

All steel work was given one coat of Goheen carbonizing coating at the shops; and two coats of the same after erection. The finishing color was black. This is expected to afford ample protection against the smoke and gases from locomotives on the tracks beneath the structure. The I-beam stringers for the roadway, however, were not oiled or painted either in the shops or after erection, for the reason that they were to be embedded in concrete, as already described. They were cleaned of rust and dirt before the concrete was placed.

LOADING.—In the loading assumed for the design, the dead load was from 5,800 lbs. per ft. for the viaduct spans to 11,300 lbs. per ft. for the Kaw River Bridge spans. The live load for the roadway was that of Waddell's specifications, Class A. The live load for the railway consisted of a continuous series of double-truck cars on each track; the trucks being 19.5 ft. c. to c., with a wheelbase of 4.5 ft., and a weight of 10 tons on each axle. The spacing of adjacent cars was 23.75 ft. c. to c. of trucks. The wind pressure was taken at 30 lbs. per sq. ft.

COLUMNS.—The viaduct columns, Fig. 6, are of H section 17½ × 15 ins. Each short side is a 15-in. steel channel with flanges outward, and between these is a ½-in. diaphragm or web plate 16 ins. wide, riveted between two pair of angles 5 × 3 ins. There are also side stay plates or batten plates spaced 3 ft. apart. The shoe is built of plates and angles, and has riveted to the channels two 18-in. sleeves for the anchor bolts. Angle brackets and plates are also riveted to the columns for the connections of the longitudinal

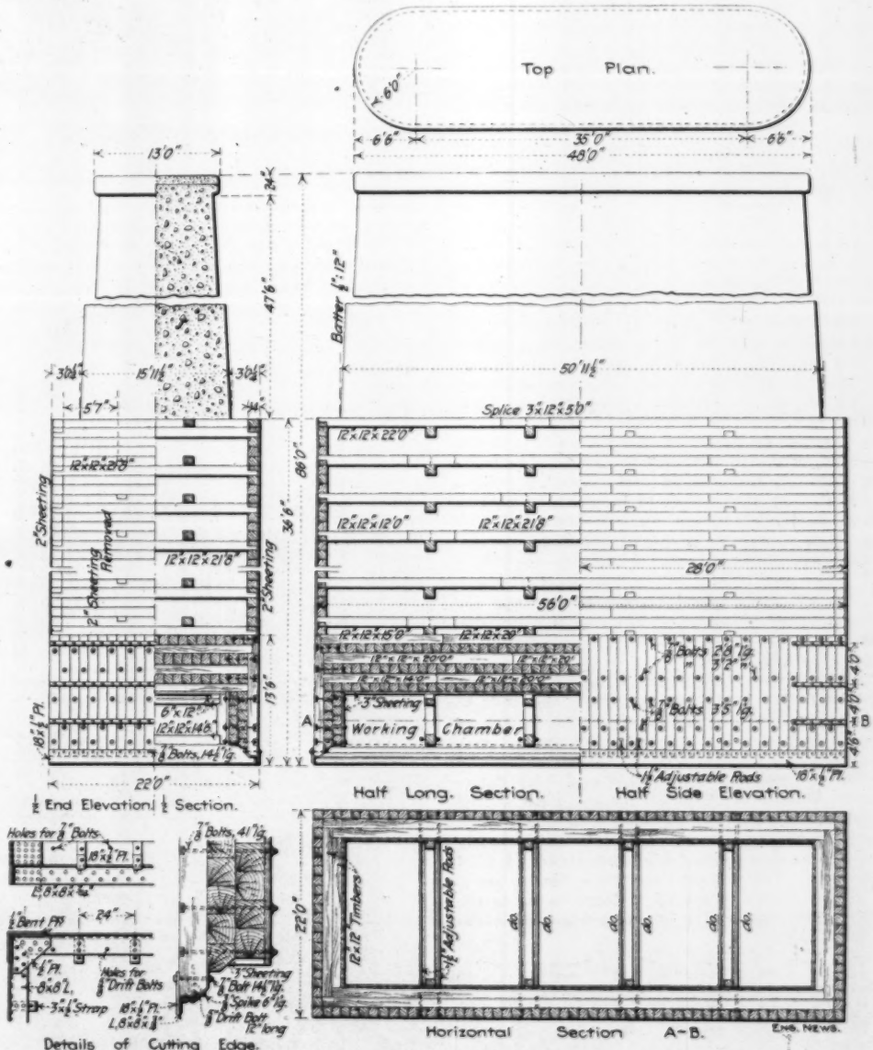


FIG. 4. CAISSON AND PIER NO. 2 FOR 300-FT. TRUSS SPAN OVER THE KAW RIVER; KANSAS CITY VIADUCT.

girders and the bracing. Fig. 6 shows the pocket for the expansion joint, and also the character of the longitudinal bracing in the 30-ft. tower spans. The columns are filled with concrete to a height of about 30 ins. above the base plate, the top of the concrete being neatly finished with ¾-in. of cement mortar.

PLATE GIRDER SPANS.—The 45-ft. girders are 6 ft. deep; they have ⅜-in. web plates, four flange angles 6 × 6 ins. and four 14-in. cover plates 24 and 34 ft. long. They are divided into 5-ft. panels by stiffeners composed of angles 3½ × 3½ ins. crimped to fit the flange angles. The web of each girder is made in two sections, spliced at the middle by two 12-in. vertical plates on the web, and two 5-in. plates in each chord. The 30-ft. girders in the tower spans are 5 ft. deep, of general construction, but the web plates are made in one length.

The cross girders or floor beams 15 ft. apart are 4 ft. 4 ins. deep, of similar construction to the longitudinal girders, but without cover plates. The ends are milled and fitted to filler plates on the webs of the longitudinal girders. Horizontal diagonal bracing is provided in the upper part of the 30-ft. spans. The cantilever girders on each side have top cover plates extending across the columns and riveted to the top flange angles of the cross girders; at the bottom of each cantilever is a cast-iron block which transmits the thrust against the side of the column. Each trolley pole passes through a hole in a plate projecting from the top flange of the cantilever girder and is seated in a cast-iron socket or a bracket riveted to the web of the girder.

TRUSS SPANS.—As already noted there are three truss spans in the viaduct, all of these being deck structures with parallel-chord riveted lattice trusses of the double intersection Warren type, with subdivided panels. In Fig. 8 are shown the general features of the 147-ft. skew span over the tracks of the Kansas City Southern Ry. The two trusses are 15 ft. 6 ins. deep, c. to c. of chords, and the panel length is approximately the same. They are spaced 33 ft. c. to c., and have cantilever girders forming extensions of the floor beams on each side, as on the viaduct. At the west end (with expansion bearing) they are supported on columns of box section, 36¼ × 32 ins. composed of eight angles 6 × 6 ins., two outside 36-in. cover plates and an interior web plate. One of these is shown in Fig. 9. At the top and bottom are two short web plates, forming a cross and secured to the main web plate by four angles at the axis of the column. The end bearings are 10-in. pins passing through the chords and cast-steel shoes. Each shoe at the expansion bearings rests upon seven segmental rollers 8½ ins. high and 39½ ins. long, carried in a special bed-plate casting. At the east or fixed end, the columns are 42¼ × 32¼ ins., and of the heavy box section shown in Fig. 9. The cast-steel shoes at this end are riveted to cap plates on the tops of the columns. The two columns at each end are connected by top struts and diagonals.

In Fig. 10 are shown some details of the truss and its rocker bents for supporting the ends of the adjacent girder spans. The chords are of trough section, while the end posts and all web members are of box section. The compression members have two web plates and four flange angles, with lacing bars at top and bottom. The tension members are pairs of channels placed with the flanges inward and connected by lacing bars. The floor beams are plate girders 4 ft. 4 ins. deep. The diagonal end floor beams are 5 ft. 6 ins. deep, and are 62 ft. long, owing to the skew. The composition of end posts and chords is as follows:

End Posts.	Top Chord.	Bottom Chord.
2 webs ¾ × 24 ins.	1 cover pl. 1-16 × 20 ins.	2 web pl. ⅛ × 30 ins.
2 Ls. 3½ × 3½ × ½.	2 top Ls. 3½ x 3 × ⅜.	4 Ls. 6 × 3½ × ½.
2 Ls. 6 × 3½ × ½.	2 web pls. 11-16 × 24.	
	2 bott Ls. 6 × 4 × ¾.	

The total weight of this span is about 375 tons. The two spans of 300 ft. 10 ins. over the Kaw River are of similar design, but with intermediate vertical posts above the intersection points, as shown in Fig. 11. Their trusses are 37 ft. 6 ins. deep c. to c. of chords, and have 20 panels of 15 ft. 0.1½ ins. These spans weigh about 900 tons each.

FLOOR SYSTEM AND PAVING.

ROADWAY.—The floor system of the roadway, Fig. 12, consists of 12-in. longitudinal I-beams spaced 5 ft. apart and resting on the floor beams or cross girders (including the cantilever girders on one side). Upon these is a floor slab of reinforced concrete, 6½ ins. thick at the middle (or 7½ ins. where the width is 38 ft.) and 5½ ins. at the sides. The reinforcement consists of ¼-in. and ½-in. corrugated steel bars laid longitudinally and transversely in both the top and bottom of the slab. All the longitudinal bars are spaced 12 ins. c to c., while the transverse bars are spaced 6 ins. in the lower row and 12 ins. in the upper row. The concrete is carried down to enclose the I-beams, as shown, and at the sides, this arrangement together with the gutter curb forms a deep girder. The concreting was done by means of a portable concrete plant mounted on a flat car and traveling on one of the street railway tracks. No wheelbarrows were needed as the machine was at such a height above the floor level that wooden troughs delivered the concrete to any part by gravity, the Smith concrete mixer being tilted to discharge the finished batch directly into the chutes or troughs. This is shown in Fig. 13.

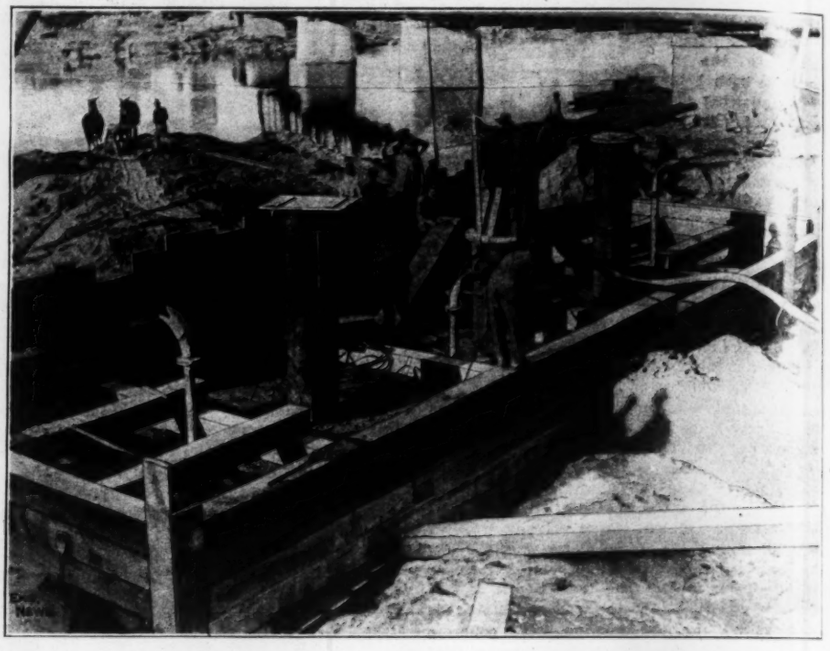
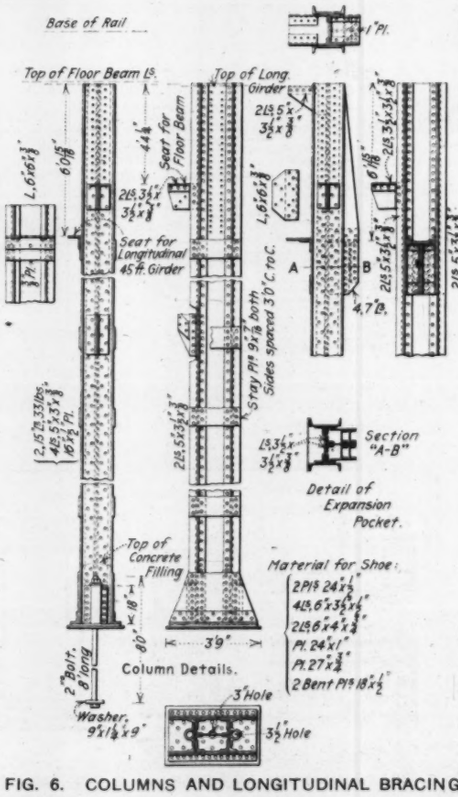


FIG. 5. SINKING CAISSON FOR PIER NO. 1 OF KAW RIVER SPANS; KANSAS CITY VIADUCT.



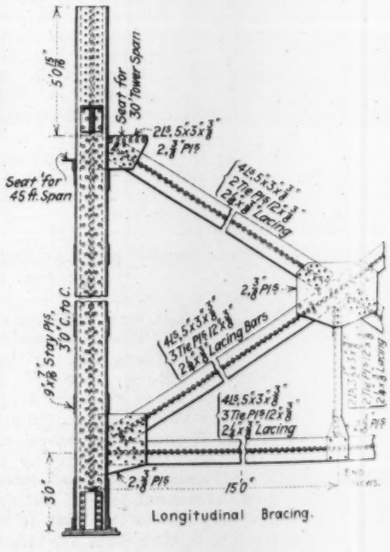


FIG. 6. COLUMNS AND LONGITUDINAL BRACING OF TOWER SPANS; KANSAS CITY VIADUCT.

The top surface of the concrete was roughened with grooves and score marks to form a bond for the 2-in. asphalt pavement. The roadway is crowned 4 ins. for the 30-ft. width (5 ins. for 38-ft. width) and has at each side a concrete gutter with grated openings to 6-in. cast-iron drains placed about 100 ft. apart. The hand railing consists of three lines of 2½-in. pipe and

a top rail of 3-in. pipe, carried in cast-iron standards or posts of rectangular section spaced 9 ft. apart. One of these is shown in Fig. 14. The pipes are keyed to the posts, except at the expansion joints. The posts are bolted against the concrete stringers by bolts to the I-beam and anchor bolts embedded in the concrete. The floor has an expansion joint in each bay of eight spans and the form of expansion joint is clearly

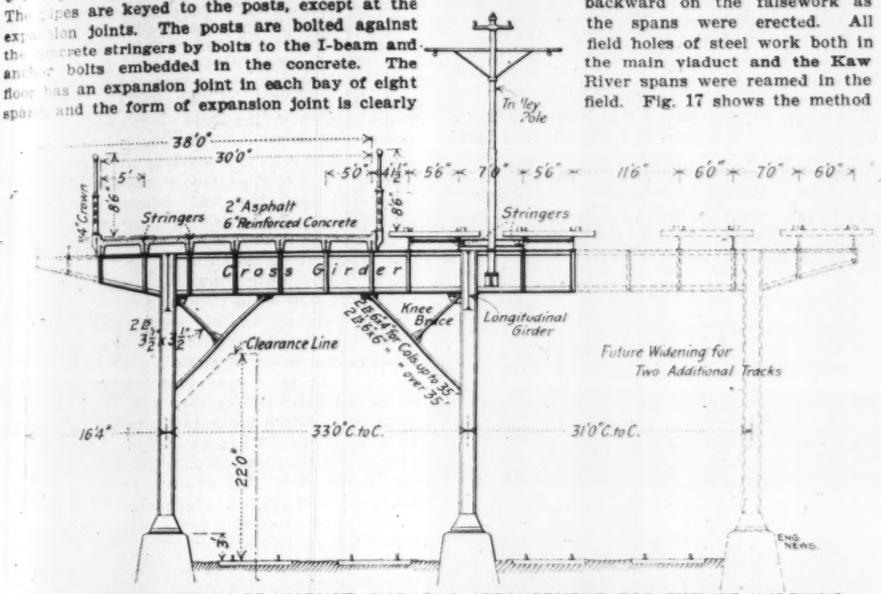


FIG. 7. CROSS-SECTION OF VIADUCT, SHOWING ARRANGEMENT FOR FUTURE WIDENING.

shown in Fig. 15. The stringers are halved and slide one upon the other, with ends normally 3 ins. apart. The concrete floor slab on each side is laid up to an angle-iron, and between these are flexible trough plates filled with a flexible asphaltic material.

TRACK FLOOR.—For the street railway tracks the floor system consists of creosoted yellow-pine ties spaced 16 ins. c. to c. and carrying 80-lb. tee rails. The ties are 6 × 8 ins., laid flat, and boxed out to 5½ ins. over the stringers; they are 9 ft. long, but every fourth tie is 22 ft. 6 ins. long, carrying both tracks. On curves, the ties are laid on edge, and boxed out to give a maximum superelevation of 1½ ins. The ties are fastened to the stringers by ¾-in. hook bolts at every alternate tie. On the ties are guard timbers of creosoted yellow-pine, spaced 6 ft. 6 ins. c. to c.; they are 6 × 8 ins. in section, boxed out 1½ ins. at the ties, and secured to every alternate time by ¾-in. bolts. Each track is provided with inside guard rails consisting of tee rails.

ERECTION.

The erection of the superstructure was in charge of Mr. John P. Wagner, Resident Engineer for the contractors, the Riter-Conley Mfg. Co., of Pittsburg, and to him we are indebted for particulars of this part of the work. The main viaduct was erected by means of a two-boom traveler, Fig. 16, consisting of two longitudinal sills, 48 ft. in length with a stiff bent consisting of fixed masts and cap to which the stiff-legs were attached. In the illustration, the hoisting tackles of both booms are attached to the eye of the hoisting stirrup on the girder. Movable masts were placed directly in front of the fixed masts, these being 14 × 14 ins. and 40 ft. long. The booms used in the traveler were 14 × 14 ins. and 50 ft. and 60 ft. long. Two Lidgerwood hoisting engines were used on the traveler and ⅝-in. steel wire cable was used for the boom and hoisting lines, so that the booms could be handled independently of each other at the same time. The traveler was 33 ft. wide at the base and was moved forward as the work progressed.

The Kaw River spans, consisting of two 300-ft. spans, were erected on falsework, as shown in Fig. 11. The trusses of these spans were 33 ft. between centers, and were erected by means of a two-boom traveler of such width as to allow a material delivery track to pass along the side between the trusses in order to deliver the material to the front of the traveler. The construction of this traveler was similar to the one above described, with the exception that it did not have a stiff bent, but rather was composed of two stiff-leg derricks. The spans were completely erected by this traveler, which was moved backward on the falsework as the spans were erected. All field holes of steel work both in the main viaduct and the Kaw River spans were reamed in the field. Fig. 17 shows the method of erecting one of the girders of the 107-ft. span adjacent to the 147-ft. truss span already described. These girders were too long to be handled by the traveler alone, and the hoisting tackle of the traveler was therefore attached near one end. At the other end a narrow gallows frame, with side braces, was erected, and a hoisting-tackle attached to its cross beam. Instead of a stirrup, as in Fig. 16, each set of tackle has a pair of clamps gripping the top flange of the girder.

ENGINEERS AND CONTRACTORS.

The firm of Waddell & Hedrick originally acted as Consulting Engineers to the Kansas City Viaduct & Terminal Ry. Co., but on the dissolution of that firm, in 1906, the work was entrusted to Mr. Ira G. Hedrick, M. Am. Soc. C. E., Consulting Engineer, Kansas City, Mo. For plans, photographs and other information we are indebted to Mr. Hedrick. The contract for the substructure, including the manufacture of the concrete piles, was awarded to Mr. James F. Halpin, of Kansas City; but the piling work was sublet to the Foundation Co., of New York and Kansas City, and the piers for the Kaw River Bridge were sublet to Kahmann & McMurray, of Kansas City. The contract for the superstructure was awarded to the Riter-Conley Mfg. Co., of Pittsburg, Pa., and Mr. John P. Wagner was their engineer in charge of the erection work. The reinforced-concrete floor was built by the Expanded Metal & Corrugated Bar Co., of St. Louis, Mo., with Mr. Geo. R. Heckle as Superintendent in charge of the work.

Construction was commenced on Aug. 31, 1905, and the entire foundation work, including all the pedestals and the piers for the Kaw River Bridge, was completed by Feb. 13, 1906. The erection of the steel work was commenced in February, 1906, and the main structure was completed (including floor and paving) by Jan. 29, 1907. The completed structure, including the laterals and all extra work, represents a total cost of about $3,300,000, or about $400 per lin. ft.

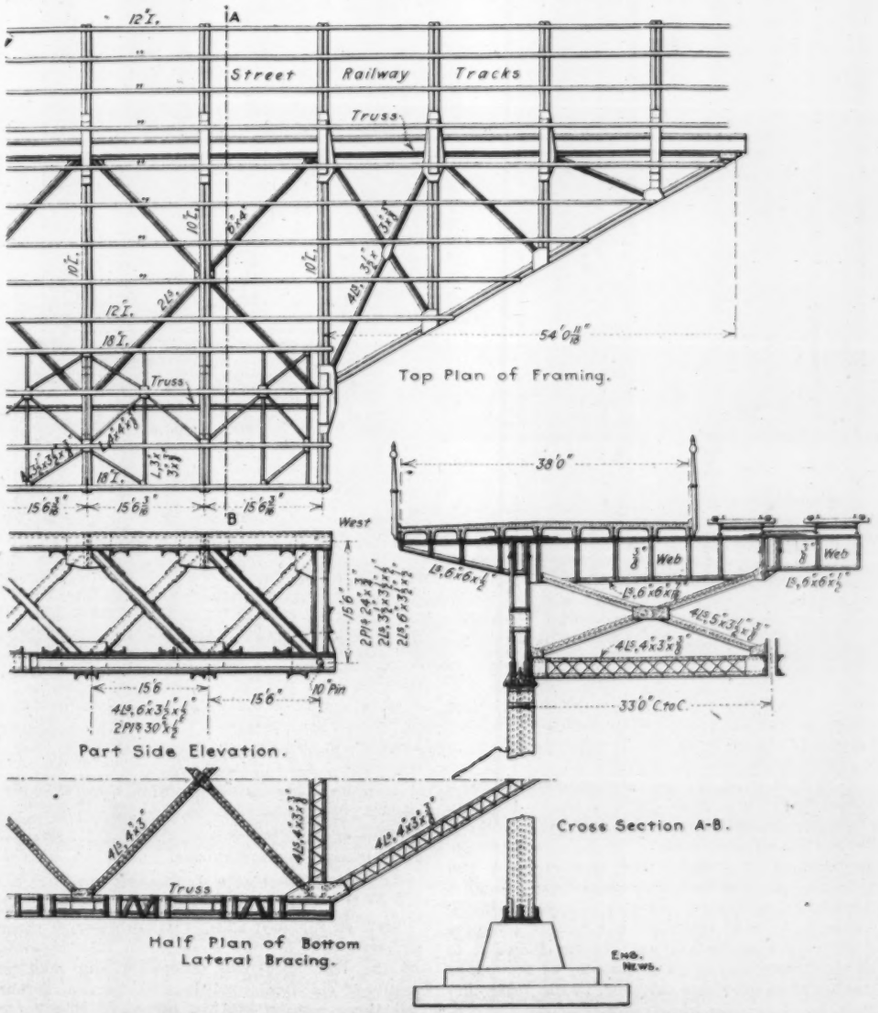


FIG. 8. 147-FT. SKEW TRUSS SPAN; KANSAS CITY VIADUCT.

THE CORROSION OF IRON: RUSTING.

GENERAL THEORIES.

Recent papers on the corrosion of iron seem to present enough contributory evidence to establish the theory that this action is fundamentally electrochemical. In Bulletin 30, of the office of Public Roads, United States Department of Agriculture, Mr. Allerton S. Cushman has presented an extended exposition of the electrolytic theory and its experimental basis. There are a few points in this treatment of the electrolytic theory that are materially cleared by studying the work of Prof. W. H. Walker, of the Massachusetts Institute of Technology, in connection with this bulletin. The interesting comparisons of various theories, the more suggestive of the experiments and applications of the electrolytic theory, from these sources are very briefly outlined in what follows. Judging from the literature of past investigations, all experimenters are agreed on two points; that iron can not rust in air or oxygen unless water is present and conversely that iron can not rust in water unless oxygen is present. Three theories have been advanced to explain the phenomenon of rusting; the carbonic-acid, the hydrogen peroxide and the electrolytic. From a superficial study these theories involve distinctly different chemical reactions, but a more complete study of the electrolytic explanation shows it to be so comprehensive as to include as secondary reactions the processes assumed, in the other theories, as of primary importance.

THE CARBONIC-ACID THEORY.—This theory is the oldest and, at the present, most widely accepted. It is assumed that rusting cannot occur without the presence of carbonic or some other acid and oxygen. The process is cyclical with three necessary factors; the acid, water, oxygen. The first reaction is given by the formula:

$$2\,Fe + 2\,H_2\,CO_3 = 2\,Fe\,CO_3 + 2\,H_2$$

Water and oxygen act on this ferrous carbonate and cause the iron to separate as ferric hydroxide, liberating the former amount of acid involved, which again acts on the metal starting the cycle over again. The secondary reaction is:

$$2\,Fe\,CO_3 + 5\,H_2\,0 + 0 = 2\,Fe\,(OH)_3 + 2\,H_2\,CO_3$$

THE PEROXIDE THEORY.—This theory presupposes the formation of hydrogen peroxide in rusting and that prevention of the formation of hydrogen peroxide must prevent rusting. The theoretical chemical equations involved are:

$$Fe + O_2 + H_2\,O = Fe\,O + H_2\,O_2$$

$$2\,Fe\,O + H_2\,O_2 = Fe_2\,O_2\,(OH)_2 = Fe_2\,O_3\,H_2\,O$$

As the second reaction accounts for the use of only part of the $H_2\,O_2$ developed by the first, two further reactions are quoted:

$$Fe + H_2\,O_2 = Fe\,O + H_2\,O$$

$$2\,Fe\,O + H_2\,O_2 = Fe_2\,O_2\,(OH)_2 = Fe_2\,O_3\,H_2\,O$$

THE ELECTROLYTIC THEORY.—This theory involves the idea of dissociation, the functions of the ions, electrical potentials and readjustments of electrical conditions in chemical actions. All these conceptions are far more complicated and less easily grasped than the purely ch... concept of the carbonic-acid and hy... peroxide theories.

For electrolysis under the conditions ... rust occurs there should be two ... metals in electrical contact and both ... tact with a liquid which is able to atta... iron and cause it to go into solution. A ... point the iron is in solution as a ferrous h... ide, which, in the presence of oxygen, is r... oxidized to ferric hydroxide and more slo... oxides, all of which precipitate as rust. It ... may be considered as a somewhat heteroge... structure, physically and chemically, the ... condition for electrolysis is filled and the ... bility of this theory rests on the possibil... pure water acting as an electrolyte. The qu... to be finally settled was whether or not ... went into solution in pure water. Arrhenius ... others* have shown that pure water is dissoci... ated and that hydrogens ions are present ... such, to the extent of about 1 gram in 10,000,000 liters. Pure water may be regarded as an acid to this extent. It was shown in Mr. Cushman's Bulletin that iron did actually pass into solution to a very limited amount. The theory of this first action between the pure water and iron was not discussed to any great extent, more space being devoted to the experiments. The reasons for the iron passing into solution and to such a slight extent, as has been found with the purest water and the purest iron, may be at once evident to persons well versed in electrochemistry, but as these points are obscure to the average practicing engineer some discussion of the entire action may be desirable.

It has been shown that metallic ions in a solution have a certain osmotic pressure and also that metals have an "electrolytic solution pressure" or a tendency to pass into solutions in the ionic form. Some metals immersed in solutions of their own salts have the osmotic pressure of the metallic ions greater than the electrolytic solution pressure so that when the metal is immersed in the solution, positively charged ions pass to the metal, causing a certain positive potential difference between the metal and the solution. Other metals immersed in solutions of their own salts have the electrolytic solution pressure greater than the osmotic pressure of the metallic ions and then metallic positively charged irons pass from the metal to the solution, causing a negative difference of potential between the metal and the solution. The osmotic pressure and the voltage developed between the metal and the solution ultimately reach a point where their combined effect limits the ionic exchange. The accompanying table by Ostwald† shows potential differences and electrolytic solution pressures of several metals immersed in solutions of their salts.

It is known that commercial irons are of a comparatively heterogeneous structure, and contain more or less segregated amounts of other metallic substances. It is conceivable that iron, being so near the critical point of Ostwald's table, when immersed in solutions may develop areas of both positive and negative potential difference from the solution itself, on account of such structural or chemical peculiarities. Here, then, would result a true electrolytic couple, short circuited. By the electrolysis about this short circuited couple iron goes into solution at the area of negative potential and hydrogen must be deposited at the area of positive potential. This deposition of hydrogen would tend to "polarize" the latter area, i. e., would prevent the contact of charged hydrogen ions. This could only result in stopping the electric current in the couple, allowing the potential at the points, where the iron was formerly going into solution, to rise to a value which would stop the solvent action.

This polarizing action of hydrogen may be quoted as an answer to the question as to why the solution of iron is so extremely limited in the absence of oxygen. Another question naturally follows this last: Why does not this ...

*Text-Book of Electrochemistry: Arrhenius; McCrae, p. 193.

†Text-Book of Electrochemistry; Arrhenius; McCrae, p. 237-238.

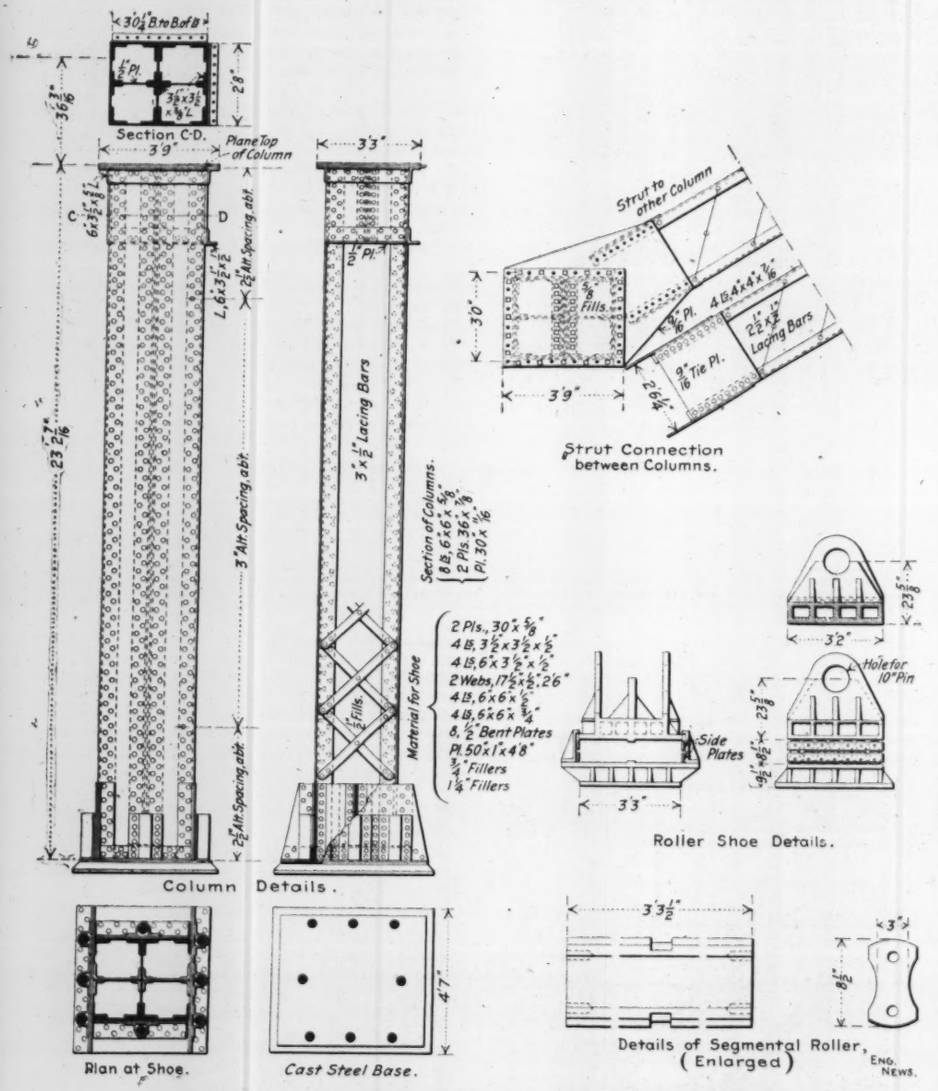


FIG. 9. COLUMNS CARRYING EXPANSION END OF 147-FT. TRUSS.

posited hydrogen similarly limit the solution of iron in the presence of oxygen; whose function is to oxidize the iron in solution to a state where it is precipitated as rust. On this last question, and on the preceding one also, interesting and

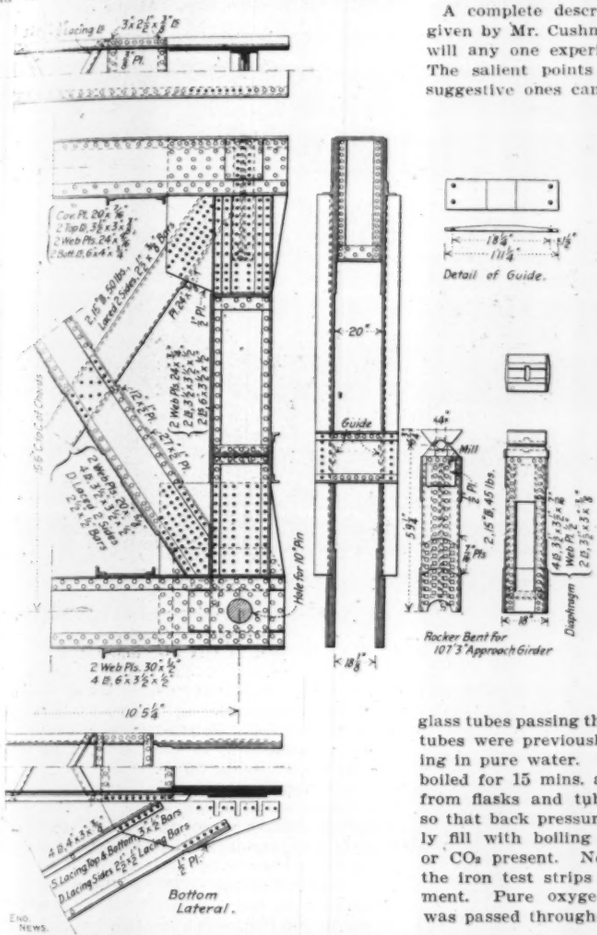


FIG. 10. DETAILS OF 147-FT. TRUSS, AND ROCKER BENT OF ADJACENT GIRDER SPAN.

enlightening matter is presented in a paper by Prof. W. H. Walker, Miss Anna M. Cederholm and Mr. L. N. Bent, before the May meeting of the New York Section of the American Chemical Society. The experimental investigations considered in this paper were carried out at the Research Laboratory of Technical Industry Massachusetts Institute of Technology.

According to these experimenters, oxygen plays a part of primary importance in the electrolysis and its functions are not limited to oxidizing the iron already in solution. This primary function of oxygen is shown experimentally to be the depolarizing of the cathodic iron areas on which hydrogen has been deposited, and the formation of water or hydrogen peroxide.

In the process of rusting it would be expected that differences of potential might be observed between the areas, on a piece of iron, where metal was being pitted and those areas more unaffected. Prof. W. H. Walker presented experimental evidences of this potential difference at the June meeting of the American Society of Testing Materials.*

OSTWALD'S TABLE OF POTENTIAL DIFFERENCE AND ELECTROLYTIC SOLUTION PRESSURE OF METALS IN SOLUTIONS OF THEIR SALTS.

	Potential difference, volts.	Solution pressure in atmospheres.
Magnesium	— 1.22	10^{444}
Zinc	— 0.51	10^{185}
Aluminum	— 0.22	10^{113}
Cadmium	— 0.19	10^{67}
Iron	— 0.06	10^{22}
Nickel	+ 0.02	10^{-9}
Lead	+ 0.10	10^{-37}
Hydrogen	+ 0.25	10^{-91}
Copper	+ 0.60	10^{-219}
Mercury	+ 0.99	10^{-361}
Silver	+ 1.01	10^{-368}

EXPERIMENTS TO ESTABLISH THE ELECTROLYTIC THEORY AND TO SHOW CO_2 AND H_2O_2 REACTIONS AS SECONDARY.

A complete description of all the experiments given by Mr. Cushman will not be attempted nor will any one experiment be completely outlined. The salient points only, of some of the most suggestive ones can barely be pointed out.

RUSTING IN THE ABSENCE OF CO_2.—Two Jena flasks and a beaker were filled with freshly distilled water and boiled for ½ hr. While the water was still boiling polished strips of charcoal iron and steel were slipped into the flasks, which were then connected to the beaker with glass tubes passing through rubber stoppers. These tubes were previously cleaned by prolonged boiling in pure water. The flasks were then further boiled for 15 mins. and all traces of air expelled from flasks and tubes. The heat was removed so that back pressure caused the flasks to entirely fill with boiling water with no trace of air or CO_2 present. No trace of rust appeared on the iron test strips kept in this sealed arrangement. Pure oxygen carefully freed from CO_2 was passed through the flasks and in less than 5 mins. rust appeared. This experiment has been frequently repeated with every possible precaution against admission of air or CO_2 and always with similar results. This experiment is given by Mr. Cushman as contributory evidence that carbonic acid is not necessary for the reaction between iron, water and oxygen.

ON THE PEROXIDE THEORY.—It has been claimed that iron placed in hydrogen peroxide is rapidly oxidized with the formation of ferric hydroxide and oxides. It is generally known that commercial hydrogen peroxide is invariably acid and impure and that it is difficult to prepare a perfectly neutral solution. However, this can be secured after two fractional distillations by taking careful precautions.

Iron immersed in a pure neutral 1% solution remained bright for a protracted period. If a flask, containing iron or steel specimens in a 1% pure neutral solution of hydrogen peroxide, be connected to a vacuum pump, oxygen is freely evolved from the surface of the metal without the least appearance of rust.

As hydrogen peroxide is more or less unstable in alkaline solutions the supporters of the peroxide theory claim that iron should not rust when immersed in such. However, rusting has been shown to take place in alkaline solutions, provided that a certain concentration is not reached. It has been shown also that rusting is actually accelerated in solutions of potassium iodide, iodine, dilute potassium permanganate and similar substances which destroy hydrogen peroxide.

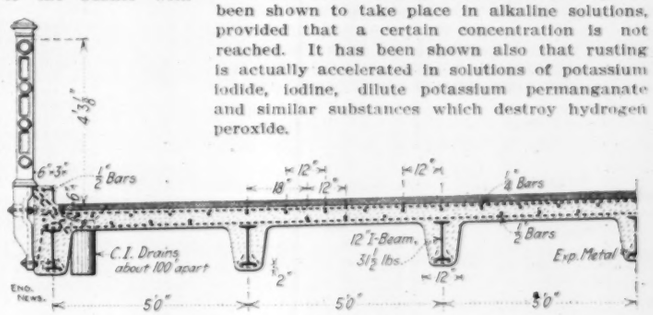


FIG. 12. CONCRETE FLOOR CONSTRUCTION OF KANSAS CITY VIADUCT.

These experiments do not preclude the formation of H_2O_2 as a secondary reaction of the electrolytic process, and are stated as contributory evidence that the peroxide reactions are not necessary, to the rusting and that a theory based on them is untenable.

IN SUPPORT OF THE ELECTROLYTIC THEORY.—The tenability of this theory was stated to depend primarily on showing that iron actually passes into solution, even very slightly, with pure water. This action was positively denied by earlier experimenters but has been lately demonstrated. Several Jena flasks were partially filled with freshly distilled water and two drops of an alcoholic solution of phenolphthalein indicator (for hydroxides) were added. By an arrangement of the apparatus similar to that

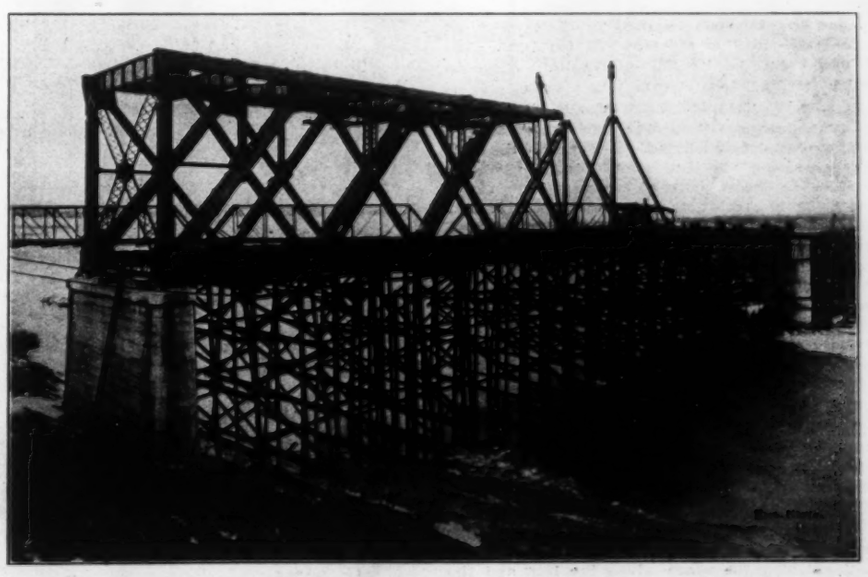


FIG. 11. ERECTION OF 300-FT. TRUSS SPANS OVER THE KAW RIVER; KANSAS CITY VIADUCT.

*Engineering News, July 4, 1907.

noted under the experiment to show rusting in the absence of CO_2 it was possible to obtain test strips of iron immersed for any desired length of time in water free from air, oxygen and carbonic acid. The presence of a hydroxide (necessarily here a ferrous hydroxide) was shown by a pink discoloration of the phenolphthalein indicator. The experiment was repeated a great number of times and in every case the pink color was developed, though in some instances very slowly, so that it became noticeable only after many hours. The experiments were always made with a "blank side" and in no case did the "blank" flasks (containing no iron test strips) show the color, indicating that the discoloration could not have been due to the action of the water on the Jena glass.

It may be pertinent to state here that phenolphthalein is a product formed by the condensation of two molecules of phenol (carbolic acid) with the anhydride of phthalic acid and is so weak an acid that it is not dissociated in solution. A netural solution of phenolphthalein is colorless but on the addition of an alkali, the corresponding salt, of the weak acid, is formed which immediately dissociates with colorless metallic ions and strongly rose-colored organic ions.

TO SHOW THE DEPOLARIZING ACTION OF OXYGEN.—In one of the series of experiments treated in the paper of Prof. Walker and others before the American Chemical Society, a jar was arranged with two concentric porous cups. The three spaces thus formed were filled with 1-100 normal solution KCl, free from O and CO_2. Two pieces of clean, pure iron joined by an iron wire were introduced into the outside and central space and the whole while boiling was sealed by a metallic cap. Such cells were kept for several hours without the solution of enough iron in either space to be detected by potassium ferricyanide. However, on extreme concentration iron could be shown to be present. It was considered by Prof. Walker and his co-workers that portions of both plates were anodic and other portions cathodic, with iron tending to pass into the solution in the ionic form at the anodes and with the cathodes polarized with a film of hydrogen.

By passing hydrogen streams through the three spaces, the boiling and sealing was omitted. After six hours no indications of iron were shown in the dilute solutions. When the hydrogen stream in one compartment was replaced by an oxygen stream, iron began to be dissolved at an appreciable rate *in the other compartment*, even though separated by the intermediate space in which the hydrogen stream was playing.

TO SHOW THE PRESENCE OF ELECTROLYSIS.—Cushman immersed iron in water containing phenolphthalein indicator, and as the rusting progressed the pink color arranged itself in zones about those portions where the iron was not going into solution and where hydroxyl ions would be expected to appear with true electrolysis. To a casual reader of Mr. Cushman's discussion, there is chance for confusion between the uses of phenolphthalein indicator to show that iron has gone into solution and to show electrolysis by indicating the zones about test strips of iron where hydroxyl ions are being developed, which are of course areas of positive potential.

The presence of ferrous ions themselves was not indicated by phenolphthalein but with potassium ferricyanide by the formation of "Turnbull's blue" at the zones where iron went into solution, which were the areas of negative potential.

An indicator which would show both positive and negative poles of the electrolytic action, by distinguishing the zones where the iron and the hydroxyl ions are developed was made by adding a trace of potassium ferricyanide to the phenolphthalein indicator in a neutral solution of gelatine which prevented too rapid diffusion of the colors. This combined reagent, called "ferroxyl," is accredited to Prof. W. H. Walker.

Polished iron specimens immersed in such a neutral reagent show in a few hours these blue and pink zones, indicating the presence of positive and negative poles of electrolytic action.

The action on iron in this gelatinous solution is one of common rusting under water. As a demonstration that rusting is universally electrolytic phenomena, it is stated that there has not been found, as yet, a specimen of iron so pure that no positive and negative areas are shown by blue and pink zones in the "ferroxyl" indicator.

ELECTROLYSIS SHOWN BY APPEARANCE OF PITTING.—According to the electrolytic theory, and as shown in the preceding experiments with "ferroxyl" indicator, when a plate of iron is immersed in water, the rusting that takes place develops positive and negative areas of greater or less extent relatively. It is possible to conceive two cases as occurring among all the possible arrangements of positive and negative areas. The first case would be that of a positive spot surrounded by an extended negative area and second would be of a negative spot surrounded by an extended positive area. At the positive areas iron will pass into solution and be rapidly oxidized to the loose colloidal ferric hydroxide and oxide which are known to migrate, under the influence of electrolytic action, to the negative polar areas. In the progress of the rusting under water it would be expected that in the first case the ferric hydroxide would be piled up in a crater with the metal eaten away at the center, while in the second case it would be expected that the hydroxide would form in a conical mass with the surrounding metal eaten away. A low power microscope clearly shows that these two formations do occur on every sheet of iron that has rusted under the water.

FIG. 13. LAYING CONCRETE FOR FLOOR OF VIADUCT.

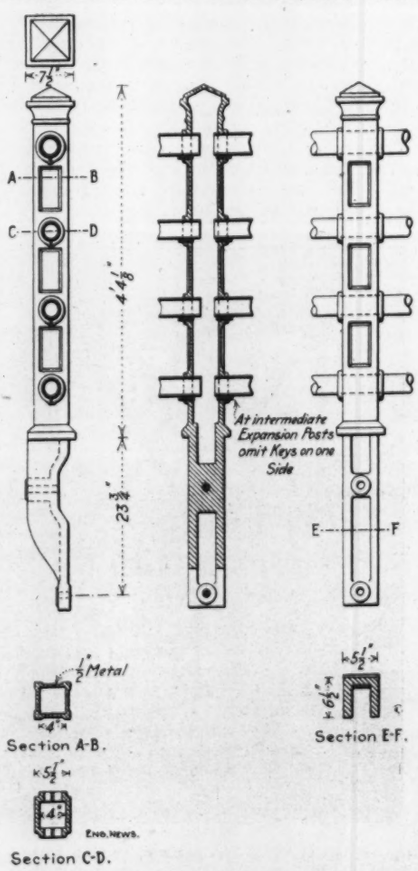


Fig. 14. Details of Posts for Hand Rail.

THE PREVENTION OF RUST.

According to the electrolytic theory, all substances in solution which contain hydrogen ions should stimulate the rusting of iron and all substances which develope hydroxyl ions in solution to a certain extent inhibit the rusting.

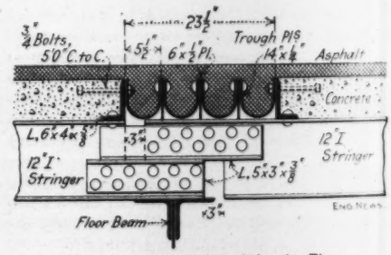


Fig. 15. Expansion Joint in Floor.

and, if the concentration is sufficient, absolutely prohibit it. This application of the theory is borne out by the behavior of acids, salts of strong acids and weak bases (which hydrolyze in solution or decompose to an acid reaction) and by the alkalies or salts of strong bases with weak acids.

It has been long known that solutions of chromic acid and potassium bichromate inhibit the rusting of iron. Experiments have shown that iron immersed in a 1-640 normal solution, or stronger, will not rust. It has been observed, too, that if a rod of bright iron or steel is immersed for a few hours in a 5 or 10% solution of potassium bichromate, is removed, thoroughly washed and wiped and is then immersed in water, rusting is inhibited for a length of time averaging from a few hours to weeks. This passive condition of the metal surface is shown in another experiment. A piece of bright steel dipped in a 1% solution of copper sulphate of 10 secs. becomes plated with a visible adherent coating of

copper. A similar piece of steel immersed ten or twelve hours in a concentrated solution of bichromate and afterwards well washed and wiped will stand from six to ten 10-sec. immersions in the 1% copper sulphate solution before an adherent visible coating is deposited. No visible change of the surface of the iron after bichromate immersion can be detected under the microscope. Cushman explains the effect as one of polarization produced by the separation of oxygen from the solution and its retention on the surface of the metal in some way without the formation of an oxide. If, as the electrolytic theory assumes, rusting is due primarily to the exchange of hydrogen ions, iron in the condition of an electrode, polarized with oxygen, should be protected from electrolytic action for a greater or less time depending on the amount of polarization. There is little doubt but that all conditions which inhibit electrochemical action also inhibit rusting of iron, which fact in itself argues for the electrolytic theory. The more the steel companies are able to produce iron from other metallic substances, and the greater the degree of homogeneity secured, the less localized, concentrated or serious will be the rusting. For the protection of present available types of iron and steel protective coating and palliative treatments are demanded. Chromic acid and its salts may come into use to inhibit rapid electrolytic corrosion such as is noticed in boiler tubes and plates. The properties of caustic alkalis to inhibit rusting may become better known and widely extended.

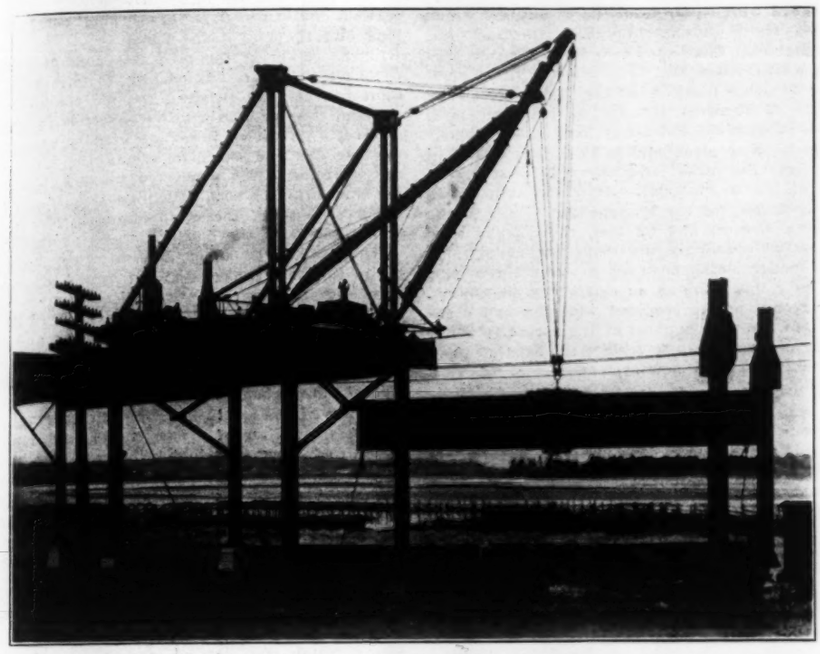


FIG. 16. TRAVELER ERECTING GIRDER SPANS OF KANSAS CITY VIADUCT.

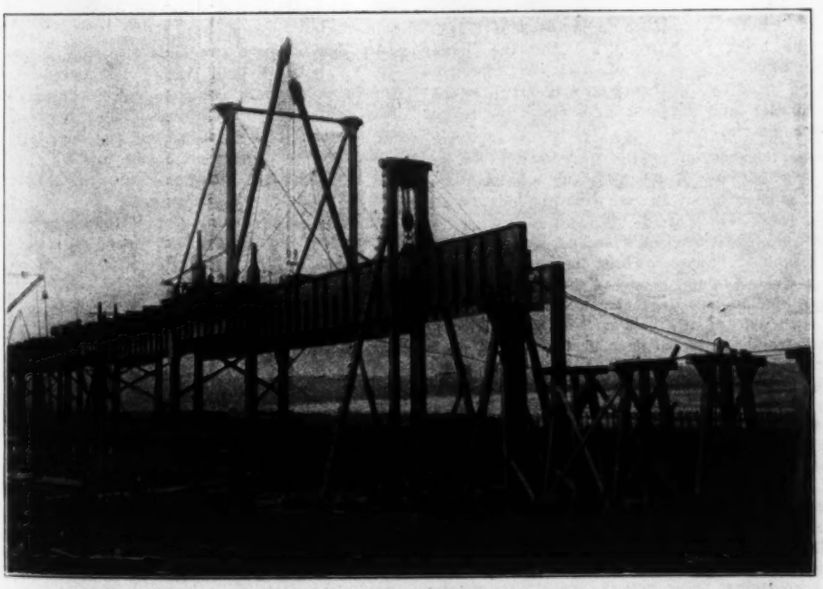


FIG. 17. TRAVELER AND GALLOWS FRAME ERECTING 107-FT. GIRDERS; KANSAS CITY VIADUCT.

AN ELECTRICALLY-DRIVEN DUMP CAR of some novelty has been brought out by the Jeffrey Mfg. Co., manufacturers of electric locomotives, etc., of Columbus, Ohio. The car is of standard design, with a steel frame; it is so constructed that an electric motor may be attached for operating it. An under-running trolley supplies current to the motor. One size manufactured has a capacity for 32 cu. ft. of material, and weighs, empty, about 2,900 lbs., including the motor and transmission gearing; the latter is a single-reduction gear, driving one axle. The car is capable of hauling a train of several trailers.

THE PAINTING OF STEEL PASSENGER CARS.*

Several hundred steel cars are now in use in passenger equipment, and the problem of the treatment for preservation and appearance is being settled temporarily by the application of paint coatings in accordance with established methods. There seems to be a demand, however, for changes in these methods, with a view to the production of a finish requiring fewer coats and less labor than are necessary with the materials now in common use.

Before any protective coatings are applied, all oil, grease, dirt, scale and rust should be entirely removed, and a priming coat applied before additional rust forms on the surface. This may be accomplished in several ways, the treatment being varied according to the condition of the metal, and the conditions under which the work is done. The oil and grease may be removed by benzine or turpentine and wiping with rags or waste. Scale and rust can be most thoroughly removed by the sand blast, or by dipping the parts in a sulphuric acid bath, and rinsing with water. Conditions may be such that certain parts cannot well be cleaned by the best methods and in such cases scrapers and wire brushes may be used to advantage. For cleaning all exposed parts, however, we strongly urge the use of the sand blast; also for interior parts that are to be coated.

In connection with the preparation of the metal we would point out the advantage of having sheets with a level surface, free from waves and roughness, for all parts that are to be finished with a view to improving their appearance. A glossy or smoothly painted finish does not hide the waves in the sheets, but tends to exaggerate them, and a surface built up by the application of putty and plaster, in order to hide roughness in the metal, is, for obvious reasons, to be avoided as much as possible. A level metal surface protected by a film of even thickness is more desirable.

During the process of construction and before they are covered up, all parts of the framing, also all parts that are hidden when the construction is completed, should be cleaned as indicated, and then covered with at least two coats of a good metal preservative of a non-inflammable nature. Where metal parts are in contact and either riveted or bolted together, they should be coated with a heavy mixture of the same character, so as to exclude moisture. For these coatings carbon and iron oxide paints are desirable. Tar and asphaltum mixtures should not be used.

In considering the treatment of the exterior, the parts may be divided into two groups: 1, The prominent parts, such as body, sashes, doors, etc., which, in addition to being protected, should also be attractive in appearance; 2, those treated mainly for their preservation, such as roof, bottom, underframing, trucks, etc., where appearance is a secondary matter. In the first case, the parts may receive several coats each of surfacer, color and varnish, in order to give them a glossy finish similar to that applied to the exterior of the wooden cars, the different coatings being applied in about the same manner as they are now used on a well-painted locomotive tank.

Among the advantages of this process may be included the knowledge (based on experience) that fairly good results can be secured; the production of a smooth and glossy finished surface that gives the equipment a well-kept appearance; and the fact that it hides much of the roughness in the metal and gives a surface that facilitates the terminal cleaning. These advantages are offset by the cost of producing such a finish; the length of time required for the work; the exaggeration of any unevenness existing in the metal sheets (by the reflection of the varnish); the practical impossibility of laying several coats of varnish on a vertical surface having numerous protruding rivet heads, in such a manner that the material will flow out smoothly, without laps or sags; and by the inflammability of the coating.

As steel has not the absorbent qualities of wood, the priming mixture may carry more pigment, and more volatile liquid, than the primer for wood; and a smaller proportion of liquid of the nature of linseed oil and varnish. The priming coat should not dry with a skin, or a high gloss, but should dry thoroughly before the succeeding coat is applied, and all the way through. The surfacing coats which follow may be applied in about the same manner as they are now applied to wood, but the glazing, or knifing fillers, which are usually applied under the rough stuff coatings should be laid over the latter. This will give them a better opportunity to dry and furnish a foundation having more virtue. While rubbing or sandpapering in oil does not produce so smooth a surface, the absence of water and decrease in time are points decidedly in favor of this method for steel cars.

In view of the fact that most varnishes perish and absorb moisture within 18 months, it would seem that the color coats should have some protective properties, in order to preserve the metal at points where the surfacer film may be thin. For this purpose we look with favor on high-grade enamels, for, if they are used in place of the flat color, each coat over the surfacer will have virtue in protecting the metal.

*Abstract of a report presented at the annual meeting of the Master Car and Locomotive Painters' Association, at St. Paul, Minn., Sept. 10, 1907.

Radical changes are sure to come, sooner or later, in the manner of treating exterior parts of steel cars. Possibly a process of plating will be found that will be effective, or a durable lacquer be produced that will maintain the metallic appearance of the steel, and at the same time protect it from corrosion. We suggest that striping and decoration might well be omitted from the exterior of the steel car. The name of the owner, numbers and other marking may be applied with raised metal letters and figures, or embossed on a plate. These changes would give an appearance of stability and make possible the use of a high-grade finishing enamel, in place of the flat color and varnish, for certain classes of equipment. Enamels do not produce so high a luster as clear varnish, but it is only under the most favorable conditions that this luster can be maintained for any length of time. Furthermore, varnish of a given quality will protect metal from corrosion for a longer period if it carries a percentage of finely ground pigment of suitable quality.

The parts of the exterior which are coated mainly for their preservation (such as roof, underframing, trucks, etc.), should be cleaned as described, and receive at least three coats of a good metal preservative coating of a non-combustible nature. For this purpose the carbon black and iron oxide mixtures are preferable to lead paints. The color should be some dark shade not far removed from the natural color of the metal.

In the treatment of the interior, the protection from corrosion is here a simple matter and the principal points involved are power to resist abrasion, sanitation and artistic effect. The metal for interior parts, if it is to be coated, should be as level and smooth as possible, in order to reduce the number of applications necessary to secure a surface that may be readily cleansed. All parts subject to a serious amount of abrasion should be so constructed that they may be easily removed and treated with baking enamels. Metal treated by this process will stand more rough usage than when coated with air-drying materials; it makes a smoother and harder surface, which facilitates the cleaning.

The headlining, which is usually of fireproof composition board, may be treated in the same manner as wood headlinings, by painting and varnishing. Other parts of the interior which are of steel construction, but not subject to serious abuse, may be coated with surfacer and rubbed down (if necessary to secure a smooth and washable surface); then coated with good enamels of the desired shade and rubbed to a finish with pulverized pumice stone in the usual way. If the finish is to be grained in imitation of wood, the surfacer should be followed with ground color, graining, varnishing and rubbing. When striping or other decoration is applied this should also be protected with varnish rubbed to a smooth finish. For all interior rubbing we believe the use of oil preferable to water. If enamels are used for finishing, they should be of the best quality, carry pigment which has been ground impalpably fine, be elastic, but dry, so that they may be rubbed to a dull finish, if desired.

The natural wood interior finish has been in use for so long that we must expect more or less demand for steel interiors finished in imitation of wood, but while it is possible to produce a good imitation by an artistic job of graining, this method of finishing is deceptive and should be discouraged. The most suitable finish for these cars would seem to be plain colors for the ceiling and upper side walls, in moderately light, harmonious tints, with a moderate amount of decoration and striping done in delicate shades to relieve the monotony. Somewhat darker colors to be used on the seats, baseboard, window sills, etc., but all to be rubbed to a smooth finish. Or a light colored ceiling with some decoration, and metallic effects on the side walls and lower parts of the car produced by painting and glazing in imitation of metal, or by the application of durable tinted lacquers to a finished metallic surface.

A LIGHT LOAD TEST of a 1,500-HP., Allis-Chalmers-Parsons steam turbine unit in the plant of the Kokomo, Marion and Western Traction Co., Kokomo, Ind., has recently been reported. The turbine is a standard horizontal type, direct connected to a 1,000-KW. alternator, intended to operate at 1,300 r. p. m. with 140 lbs. steam pressure (dry saturated) and 28 ins. vacuum (referred to 30 ins. barometer at exhaust nozzle). The maximum load possible to be applied to the generator was at no time much over 50% rated capacity. The following table shows very interesting results. As the steam input and electrical output were measured, the HP. figures are estimated.

Average electrical load	556.3 KW.
Average mechanical load on turbine assuming 93% generator efficiency	596 HP.
Per cent. of rated load	55.3%
Duration of test	4.0 hrs.
Steam pressure, turbine throttle	136.4 lbs. gage
Steam pressure, turbine inlet	61.9 lbs. gage
Vacuum at exhaust	26.59 ins.
Barometer	28.93 ins.
Vacuum referred to 30 ins. barometer	27.66 ins.
R. P. M.	1,800
Moisture in steam	2.82%
Dry steam supplied per hr.	12,115 lbs.
Actual consumption of dry steam per KW.-hr.	21.9 lbs.
do. per HP.-hr., assuming 93% generator effy.	15.4 lbs.

COST AND METHODS OF CONSTRUCTION OF TUNNEL FOR WATER PIPES UNDER MYSTIC RIVER, BOSTON, MASS.

By CALEB MILLS SAVILLE, M. Am. Soc. C. E.*

Before the construction of the Metropolitan Water-Works, East Boston and Chelsea were furnished with water from the Cochituate works of the City of Boston, through two lines of pipe, running from Charlestown to Chelsea in or parallel with Chelsea St. In two places these pipes crossed under portions of the Mystic River by means of inverted siphons of cast-iron pipe, boxed in concrete and heavy hard pine timbers.†

One of these pipes, laid in 1850, was 30 ins. in diameter; the other, laid about 14 years later, was 24 ins. in diameter. After the 42-in. main had been laid for the Metropolitan Water-Works through Everett and Chelsea, these pipes were no longer absolutely necessary for supply. It was thought wise however to maintain a pipe line over this route as an emergency connection. On account of the proposed widening and deepening of the river channel at the northerly crossing it was necessary to replace the existing pipes at this place by a new system.

As the line was now to be maintained only as an auxiliary connection, the diameter was reduced to 24 ins. Several methods of making the crossing were under consideration, but on account of the width and depth necessary to meet present and future requirements it was decided to build a tunnel under the channel with shafts on either side. A contract for the work was made with Mr. Chas. A. Haskin of Boston and work was begun Aug. 20, 1900. The tunnel and shafts are of circular section, 6 ft. in diameter in the clear, and lined throughout with 12 ins. of brick work laid in cement mortar. The shafts are 139 ft. c. to c., and are about 63 ft. deep. Each shaft is surrounded with a fender guard in the shape of a septagon, containing 14 main oak piles well driven into the river bottom. The piles in these guards are well braced and tied together and capped with two sets of 6 × 12-in. hard pine girder caps at high and low water lines. About 45 ft. of each shaft is composed of steel cylinders 8 ft. in diameter and ½-in. thick and lined with brick masonry. It was found most convenient to build the first 28 ft. of these cylinders before they were put in place and this work was done about a mile from the tunnel. The cylinders were set up on the deck of a lighter; the brick work completed and the lighter then towed to the tunnel, and the completed section lifted by the shears on a powerful wrecking lighter and dropped into place inside the fender guards.

Near the bottom of the first section set up, there was fastened to the walls a well-braced circular shelf about 12 ins. wide and on this the brick work was started. When the first section was completely bricked a second was bolted on by means of flanges. These flanges were angle irons riveted to the inside of the cylinders. Between the flanges were placed two gaskets, one on each side of the bolt circle. These gaskets were strands of tarred yarn braided in flat plaits and soaked in oil and red lead. When three sections of cylinder had thus been bolted together and completely bricked it was floated to place as previously stated. The total weight of this portion was about 40 tons. After it was lowered into place and had settled somewhat into the river bottom, additional steel cylinders were bolted on and 12 ft. of brickwork added. In the meantime, the working plant had been set up. This consisted of two air compressors, each with a capacity of 450 cu. ft. of free air per min., at 75 revolutions, three boilers, several pumps for removing water, a steam hoitsing winch, an hydraulic pressure pump and other miscellaneous machinery.

A steel plate with a circular hole 4 ft. in diameter was bolted over the top of the shaft, and on this was fastened an iron collar. On top of the collar was fastened the air lock by which the shaft was to be entered after the air pressure was put on. Over the top of the shaft a framed scaffolding was erected for hoisting material. The wire rope carrying the hoisting bucket ran from the winch, over a wheel on the scaffold and through a brass plug ten inches long into the top of the lock.

In the center of the plug was a stuffing box packed with square flax packing to form an air-tight connection. Between the air compressors and the receiver was a stiff connection, while a piece of 3-in. rubber hose was used between the receiver and the shaft to supply the air. All the machinery was connected in such a manner that either or both boilers could be run with either or both of the compressors, besides supplying steam to the hoister.

When all the machinery was in order the water in the shaft was forced out by the air pressure and the work of excavation begun in the shaft on the Chelsea side of the river. All of the excavated material was hoisted out and all of the building material taken in by means of the steam winch. The shaft and tunnel were lighted throughout by electricity taken from the wires of the local company. After excavation was begun the top of the shaft was loaded with about four tons of railroad iron and the total weight caused a settlement of about 8 ft. At this point no further settlement could be detected, even after the compressed air was taken off. The sides of the excavation as it proceeded were then completely lined with circular wooden lagging. This circular shape was carried down about 12 ft. and was then gradually enlarged and changed in form so that 2½ ft. lower it became square, the sides being about 10½ ft. long. This enlargement was made in order to get room to set up the shield and start it into the heading.

In the circular portion of the shafts and throughout the heading, as fast as the material was excavated, the sides were lined with the wooden lagging mentioned above. This was cut from 2-in. spruce plank to an inside diameter of 8½ ft., the outside diameter being 10 ins. longer, and eight pieces were required to make the full circle. Each piece was nailed to that directly back of it by four 5-in. wire nails, the sections in each row breaking joints with those immediately behind.

By this means a comparatively solid shell was quickly built to hold the outside material until the brick masonry could be put in. Excavation was begun Sept. 26, but proceeded very slowly on account of the nature of the material encountered—a coarse sand and gravel, containing many stones, quite loose. It was thus difficult to hold the air pressure.

For the week ending Oct. 6 the progress and actual cost of excavation was:

Labor	$353.50
Carting away of material	95.70
Cement washing inside of shaft	15.40
Watchman	24.50
Supplies	21.50
Carpenters	15.40
Miscellaneous expenses	40.35
Total	$568.15
Number of cubic yards excavated	69
Cost per cu. yd	$8.23
Rings of 2-in. lagging placed	55.00
Cost per lin. ft. of excavation	$1.95

The number of rings of lagging placed represents the progress of the excavation, and when it is considered that these rings were only two inches thick, it appears that the average progress per day was a little less than 16 ins. When the excavation had proceeded thus far great difficulty was experienced in holding the air pressure. Boils and bubbles appeared in the water about the shaft to a distance of 50 ft. away. In order to hold the pressure at all and so prevent the inrush of water it was necessary to keep the sides of the excavation constantly plastered with clay and as fast as the lagging was placed it was covered in the same way and afterwards washed with thick cement grout until it was air tight. The air pressure maintained varied with the height of the tide and depth of the excavation. The final scale used by the en-

*Formerly Division Engineer Metropolitan Water-Works.

†For description of the removal of these siphons see Eng. News, Feb. 28, 1901, p. 147; and for description of the building of similar siphons see Journal of the Association of Engineering Societies, February, 1886, and March, 1901.

ranged from 24 lbs. per sq. in. with the tide at Boston City datum to 29 lbs. per sq. in. with the tide at 12 ft. above this datum. To regulate the pressure a tide gage was set up and a scale of pressures posted near the compressors. The regulation of the air pressure was the most important part of the work, for if the pressure ran too high the material was so unstable that a serious blowout would occur, while if the pressure became too low there would be an inrush of water. In either event the tunnel would be flooded and the lives of the men endangered.

A great deal has been written by theorists and also by men with considerable experience in practical pneumatic work concerning the danger of entering and leaving a chamber where the pressure above the normal is maintained, unless the transition is made very slowly. Without attempting argument, or at all belittling the danger that may obtain under high pressures, the writer wishes to call attention to the air-lock practice on this work. During all of the work the men were employed in two shifts of ten hours each, leaving the shaft and tunnel for the customary mid-day and mid-night hour for lunch. It was their custom and that of the contractor, who was highly experienced in this class of work, to enter and leave the chamber very quickly, taking only a very few minutes (possibly two or three and usually less). No serious case of the caisson disease or "bends" appeared at all and only two or three of the men experienced any ill effects. Most of the men remained at work from start to finish and there seemed to be no more changing of men or loss of time from illness than is found in outside work. It seems that if care is taken to select good, sound men, and if the men themselves are not addicted to drink and weakened thereby, no serious results from this class of work need be apprehended under such conditions as were here found.

When the excavation of the shaft was finished the shield was sent through the lock in 24 pieces and assembled at the bottom of the shaft for working it into the heading.

The heading as excavated by means of this shield was approximately circular in section, having a diameter of about 9 ft. The shield was built up of steel plates and structural shapes and was about 4½ ft. long. It was forced ahead by means of hydraulic jacks, the material being excavated ahead and the wooden lagging closely following behind. As fast as eight or ten feet of section was ready it was lined with brick and well washed with cement grout. Only one serious accident occurred and that only to the work, causing a week's delay and considerable expense. When the heading was about 26 ft. from the Chelsea shaft a blowout occurred and the tunnel was filled with water. After several attempts the hole was at length successfully plugged and the work continued.

Great difficulty was experienced in keeping the shield to line and grade on account of the character of the materials encountered. Sometimes large boulders that would cause the shield to deflect as it was being forced ahead, at other times pockets of soft material into which the shield would settle. On this account the use of the shield was discontinued about 30 ft. from the Charlestown shaft and the remainder of this work done with poling boards.

The average rate of progress in the shaft was about 3 ft. in 24 hours, and in the heading about 1.8 ft. of completed work was done in the same time.

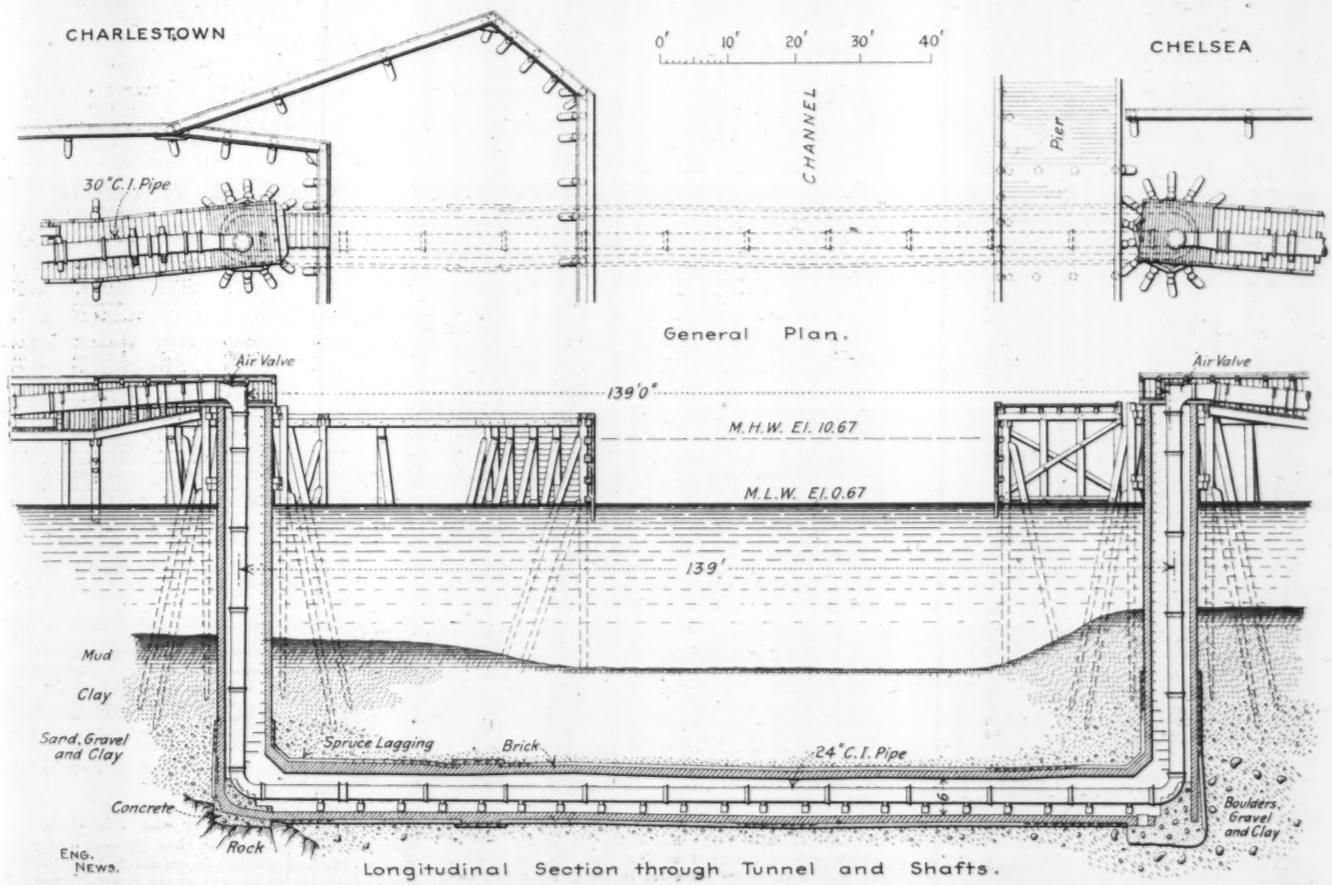
When the work was completed a 24-in. cast-iron water main of the ordinary pattern was placed in the tunnel and connected at either end to existing pipes. This work was done by day labor by the Metropolitan Water-Works force.

Before acceptance a test for leakage was made under normal atmospheric conditions and it was found that there was an infiltration of about 725 gals. for 24 hrs.

In Sept., 1903, an inspection of the tunnel and pipe line was made by the maintenance department in order to see if there had been any damage by electrolysis.

A 3-in. pulsometer was hung in the tunnel shaft on the Chelsea side and a 20-HP. boiler, hired for the purpose, was used to furnish steam. This boiler was barely able to furnish sufficient steam. The pumping was started at 4 p. m., on the 15th, and after several interruptions the tunnel was cleared of water by 3 p. m. of the next day, and kept clear during the inspection with no difficulty. The inspection showed the pipe line in good condition with a few small leaks. There were also a few leaks in the tunnel, all small. The pipe line, when it was laid, was bolted together in the shafts with 1½-in. bolts reaching from the curves at the top to that at the bottom. These rods showed decided signs of corrosion, being pitted ¼-in. deep in some places, by electrolytic action.

The work was a portion of the construction work for the Metropolitan Water-Works, Boston, of which Mr. F. P. Stearns, M. Am. Soc. C. E., was Chief Engineer; Mr. Dexter Brackett, M. Am. Soc. C. E., Engineer of the Distribution Department, and Mr. Geo. E. Wildo, Asst. Supt. in charge of Maintenance Work.



TUNNEL UNDER MYSTIC RIVER, BOSTON, MASS., FOR METROPOLITAN WATER-WORKS.

ITEMIZED COSTS.

There follows an itemized account of the cost of the work. First is given the value of the contractor's plant and following that the separate costs for each shaft and the heading with the proper percentage, to provide for deterioration, assigned to each of the three subdivisions:

GENERAL PLANT CHARGES:

Machinery:	
Air compressor—Sergeant-Colling-worth Co.	$1,800.00
Air compressor—Ingersoll	1,400.00
Boiler, 50-HP., locomotive	350.00
Boiler, 20-HP., upright	250.00
Boiler, 20-HP., upright with dd. engine	700.00
Engine, single drum hoisting	300.00
Pump, 6-in. Knowles	300.00
Pump, 4-in. Knowles, duplex	300.00
Lock and collar—double, horizontal	850.00
Lock under air receiver	200.00
Diaphragm	37.67
	$6,487.67

GENERAL PLANT CHARGES: (Continued.)

Small supplies:		
Valves	$196.60	
Tools	290.03	
Hose	163.95	
Fittings	319.57	
Wrought-iron pipe	263.17	
		$1,313.32
Setting up and taking away plant:		
Teaming	$226.80	
Labor	793.44	
Stock—lumber, nails, etc.	191.10	
		$1,211.34

STOCK AND LABOR COSTS:

Plant (charged to excavation in heading only):		
Pump—hydraulic pressure	$250.00	
Shield—original cost, 6 ft. diameter	700.00	
Fittings—brass and copper pipe, valves, etc.	128.70	
Jacks—6 hydraulic	672.00	
Valve block (for controlling shield)	107.85	
		$1,858.55
Chelsea shaft (Sept. 9 to Oct. 23, 1900—46 days):		
Fender guard (contract), complete	$1,400.00	
Setting shaft (contract)	300.00	
Use of lighter, 14 days, at $15	210.00	
Steel casing, 29,680 lbs., at 3 1/16 cts	1,020.21	
Bolts, washers and gasket	26.20	
Coke, 102.34 tons, at $3.75	383.78	
Electric lights, 32 days, at $4	128.00	
General foreman, 46 days, at $5	230.00	
		$3,698.19
Excavation:		
Labor:		
Engineers $240.10		
Laborers 848.70		
Teaming 226.85		
	$1,315.65	
Stock:		
Lumber and lagging $179.34		
Spikes 5.70		
Candles 21.00		
	206.04	
		$1,521.69
Concrete:		
Labor:		
Engineers $14.00		
Laborers 75.35		
	$89.35	
Stock:		
15 bbls. cement 39.75		
1 d. load sand 2.00		
2 d. loads cut stone 8.00		
	49.75	
		$139.10
Brick:		
Labor:		
Engineers $80.60		
Masons 124.50		
Laborers 231.60		
	$437.70	
Stock:		
15 M. brick, at $7.50 $92.50		
17.5 M. brick, at $10 175.00		
105 bbls. cem., at $2.65 278.25		
15 d. l. sand, at $2 30.00		
	575.75	
		$1,013.45
		$6,372.43
Charlestown shaft (Jan. 12 to Feb. 16, 1901—34 days):		
Fender guard (contract —)	$1,400.00	
Setting shaft	300.00	
Lighter, 6 days, at $15	90.00	
Steel casing, 32,855 lbs., at 3 1/16 c.	1,129.37	
Bolts, washers, etc.	30.00	
		$2,949.37
Coke, 42.2 tons, at $3.75	$158.25	
Coal, 11.5 tons, at $4.25	48.88	
Electric lights, 26 d., at $4.25	110.50	
General foreman, 34 d., at $5	170.00	
		3,437.00
Excavation:		
Labor	$642.75	
Stock:		
Lagging and spikes $109.20		
Candles 7.50		
	116.70	
		759.45
Concrete:		
Labor	$100.00	
Stock:		
15 bbls. cem., at $2.65 $39.75		
3 d. loads sand 6.00		
3¼ d. loads stone 13.12		
	58.75	
		158.75
Brick:		
Labor	$434.85	
Stock:		
30,850 brick, at $10 308.50		
16 d. loads sand 32.00		
102 bbls. cem., at $2.65 270.30		
	610.80	
		1,045.65
		$5,499.85
Heading (Oct. 24, 1900, to Jan. 12, 1901—71 days):		
Making over shield, 6′ diam. to 9′ 2″ diameter	$347.50	
Cross beams for shield	86.20	
Coke, 339.77 tons	1,274.13	
Electric lights, 71 days	312.00	
General foreman, 71 days	355.00	
Cleaning up:		
Labor	451.10	
Stock	23.00	
Blowout:		
Labor	458.25	
Stock	48.00	
		$3,355.18
Excavation:		
Labor	$3,397.65	
Stock:		
Lagging & lumber $928.50		
Small supplies 68.15		
	996.65	
		4,394.30
Brick masonry:		
Labor	$1,250.00	
Stock:		
Small supplies $3.50		
Brick, 64 M. 540.00		
Cement, 225 bbls. 596.25		
Sand, 29 loads 58.00		
Centers 17.18		
	1,315.02	
	2,565.02	
		$10,314.50

SUMMARY AND APPORTIONMENT OF GENERAL EXPENSES:

Heading and shafts:		
Machinery 6,487.67 at 15%		$973.15
Small supplies 1,313.32 at 25%		328.33
Setting up and taking down plant		1,211.34
		$2,512.82
Heading only:		
Machinery $1,858.55 at 15%		$278.78
Chelsea shaft,	46 days = 30% of $2,512.82	$753.85
Charlestown shaft,	34 days = 22% of 2,512.82	552.82
Heading	71 days = 48% of 2,512.82	1,206.15
Heading		278.78

These results are summarized in the "Table Showing Costs per lin. ft. of Mystic River Tunnel."

THE EFFECT OF CHANGES IN CANAL CROSS SECTIONS UPON RATE OF FLOW.

By F. W. HANNA.*

In Engineering News for June 6, 1907, under the title of "The Effect of Changes in Canal Cross-Sections Upon the Rate of Flow," the writer gave some formulas for determining the effect of canal reducing sections on the hydraulic head of the canal. Evidently some of these effects are dependent on the form of reducing section, of which there may be an endless number. The problem solved completely in the article above referred to is based, as there stated, on the assumption that the depth of water in the reducing section is constant, and that the side slopes thereof and of the canal both above and below the reducing section are planes making a constant angle with the surface of the canal bed. While this type of reducing section is common in practice, yet the irrigation engineer is very often called upon to solve the related problem based on the assumptions that the depth of water in the reducing section is constant, that the side slope planes of the canal above the reducing section and those of the canal below the same make different angles with the surface of the canal bed, and that the side slope surfaces of the reducing section are warped to conform to those of the adjoining canal stretches. It is the object of this paper to develop similar formulas for this related problem. For this purpose let the following symbols have the significance below assigned them:

c = Kutter's coefficient.
a = Water depth in feet in reducing section.
g = Gravity.
h_f = Friction head in feet in reducing section.
L = Distance in feet of any cross-section of reducing section from its upper end.
L^1 = Length of reducing section in feet.
h_v = Difference in feet between velocity heads above and below reducing section.
q = Canal capacity in second-feet.
r = Hydraulic radius in feet of cross-section of reducing section at distance L from its upper end.

*Engineer, U. S. Reclamation Service, Washington, D. C.

TABLE SHOWING COSTS PER LIN. FT. OF MYSTIC RIVER TUNNEL.

	Chelsea shaft, 67 lin. ft.	Charlestown shaft, 64 lin. ft.	Heading, 132 lin. ft.	Total.	Cost per lin. ft.*
Stock and labor	$6,372.43	$5,400.00	$10,314.50	$22,087.78	$84.00
General expense	753.85	552.82	1,484.93	2,791.60	10.60
Total	$7,126.28	$5,953.67	$11,799.43	$24,879.38	$94.60*
Cost per lin. ft.	106.20	93.00	89.40		

*Average cost of total length tunnel and shafts.

TABLE SHOWING COMPARATIVE STATEMENT AND UNIT COSTS—MYSTIC RIVER TUNNEL.

Location of work.	*No. cu. yds.	Labor Cost—Amount.	Labor Cost—Per cu. yd.	Stock Cost—Amount	Stock Cost—Per cu. yd.	†Miscel. Items—Amount.	†Miscel. Items—Per cu. yd.	Total Cost—Total amount.	Total Cost—Per cu. yd.
EXCAVATION.									
Shaft 1	70	$1,315.65	$18.79	$206.04	$2.94	$1,041.00	$14.87	$2,562.69	$36.60
Shaft 2	60	642.75	10.71	116.70	1.95	675.00	11.27	1,434.45	23.93
Tunnel	200	3,397.65	16.99	996.65	4.98	4,263.11	21.32	8,657.41	43.29
Totals and average	330	$5,356.05	$16.23	$1,319.39	$4.00	$5,979.11	$18.12	$12,654.55	$38.34
BRICK MASONRY.									
Shaft 1	51	$437.70	$8.58	$575.75	$11.30	$390.00	$7.66	$1,403.45	$27.56
Shaft 2	50	434.85	8.67	610.80	12.21	307.60	6.13	1,352.65	27.01
Tunnel	107	1,250.00	11.69	1,315.02	12.30	577.00	5.41	3,142.02	29.40
Totals and average	208	$2,122.55	$10.22	$2,501.57	$12.02	$1,274.00	$6.13	$5,898.12	$28.37
CONCRETE MASONRY.									
Shaft 1	18	$89.35	$4.96	$49.75	$2.76	$64.63	$3.59	$203.73	$11.31
Shaft 2	20	100.00	5.00	58.75	2.94	58.45	2.92	217.20	10.86
Tunnel	..	.....	....	.....	.....	.....	....	.....	.....
Totals and average	38	$189.35	$4.98	$108.50	$2.86	$123.08	$3.34	$420.93	$11.08
Grand total	‡257	$7,667.95	$29.83	$3,929.46	$15.28	$7,376.19	$28.71	§$18,973.60	$73.82

*The number of cu. yds. is based on the estimated quantity as shown on the contract plan with no allowance for additional work either in excavation for material running in from outside the neat or in brick masonry for filling cavities outside the lines.
†Includes proportional cost based on time employed in the several portions for coke, electric lighting, general foreman and allowance for cost and use of plant.
‡Total length (lin. ft.) tunnel and shafts, and unit costs per lin. ft.
§To this is to be added $5,905.78, the cost of steel castings, etc.

MYSTIC RIVER TUNNEL.

BRICK MASONRY (LAID IN PORTLAND CEMENT MORTAR.—1 ATLAS CEMENT; 1 MEDIUM SAND.)

	Data. Lin. ft.	Cubic yards masonry.	Brick M.	Cement bbl.	Deductions. Brick used. per cu. yd.	per bbl. cem.	Cement used. per cu. yd.	per M brick.	Remarks. Cu. yd. Masonry per M brick.	per bbl cem.
I	22.0	18.4	10.0	34	545	293	1.86	3.40	1.84	0.54
II	13.1	10.7	5.5	18	515	306	1.69	3.28	1.94	0.59
III	124.0	101.	63.3	207	620	306	2.03	3.28	1.60	0.48
IV	131.0	107.0	64.0	225	600	285	2.10	3.52	1.67	0.48

REGULATIONS GOVERNING THE REPORTING of railroad accidents were issued by the Public Service Commission of the State of New York, Second District, on Sept. 3; they are embodied in the Commission's circular No. 17. The regulations are addressed "to all railroad and street-railroad corporations"; in them the various species of accidents which are possible are divided into two main classes—those to be reported by telegraph and those to be reported by mail.

Under the first head are grouped (a) all accidents resulting in loss of life to passengers or employees; (b) all accidents occurring at grade crossings, resulting in death or serious injury to any person; (c) all derailments of passenger trains, or of locomotives or cars in passenger trains; (d) all collisions involving freight or passenger trains, whether resulting in loss of life or not; (e) all explosions of locomotive boilers, and all accidents to locomotive boilers resulting in death or serious injury to any person; (f) all bridge failures.

For making reports by mail a very complete form is provided—Form No. 25. Minor accidents only are to be reported simply by mail; but all accidents, whether previously reported by telegraph or not, are to be reported by mail on this form.

With the data and information received in this manner from the common carriers of the state, the Commission "aims to make its investigations as thorough as possible with a view to the prevention of accidents in the future." The instructions are very explicit, and should result in a uniformity of presentation of data that will notably decrease accident figures, ultimately, in New York State.

v = Mean velocity in feet per second in reducing section at distance L from its upper end.

v_1 = Mean velocity in feet per second in canal above reducing section.

v_2 = Mean velocity in feet per second in canal below reducing section.

w = Canal bottom width in feet above reducing section.

ϕ = One-half of angle between converging sides of bed of reducing section.

θ = Angle of side slopes at distance L from upper end of reducing section.

θ_1 = Angle of side slopes in canal above reducing section.

θ_2 = Angle of side slopes in canal below reducing section.

y = Rate of change in mean width of cross-section of reducing section per unit distance from upper end thereof produced by warping side slopes.

x = Rate of change in wetted perimeter of cross-section of reducing section per unit distance from upper end thereof produced by warping side slopes.

The formulas as developed will be stated for the case of changing from a larger to a smaller canal cross-sectional area as shown in the accompanying figures; but they are equally applicable to the converse case by making appropriate changes in them as hereinafter indicated. For the case in question there will be an increase in velocity head due to the decreased cross-sectional area and a loss of head due to friction in the reducing section. There will be but little, if any, loss of head due to contraction of canal cross-section where a reducing section with warped side slopes is used (see Treatise on Hydraulics, by Mansfield Merriman, p. 177), as this arrangement practically eliminates subsequent enlargement of the stream.

The change in the velocity head at marked changes in canal cross-sections is always one of the most important items for consideration. The gain or loss of velocity head in a reducing section is independent of the form of this section, being dependent solely on the mean velocities at the initial and terminal ends thereof. The value of this quantity is always, therefore, determinable from the following well-known formula:

$$h_v = \frac{1}{2g}(v_2^2 - v_1^2) \quad \ldots\ldots (1).$$

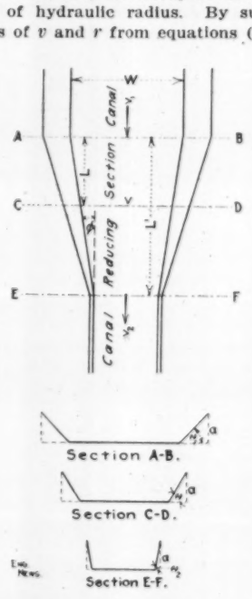
In building warped surfaces for the purpose here in question it is customary to make rectilinear the horizontal projection of the intersection of the assumed water surface with each side slope of the reducing section. In other words, the change in the mean width of the wetted cross-sectional area produced by warping the side slopes is made directly proportional to the distance of that cross-section from the initial end of the reducing section. The unit rate of change of this mean width, due to the cause in question, may, therefore, be expressed in terms of the foregoing symbols by the following formula:

$$y = \frac{a(\cot\theta_1 - \cot\theta_2)}{L^1} \quad \ldots\ldots (2).$$

It is now necessary to find the corresponding unit rate of change in the wetted perimeter of the cross-section of the reducing section produced by warping the side slopes. Strictly this unit rate of change is a function of cosecant θ, a troublesome variable in the final reduction of the friction formula. Practically this difficulty may be avoided by assuming that the change in the wetted perimeter at any cross-section of the reducing section due to warping the side slopes is directly proportional to the distance of that cross-section from the initial end of the reducing section. If the reducing section width is large compared with the water depth, the error of this assumption is very small for the usual range of variation of θ. If the channel is narrow compared with the water depth, accuracy may be increased by computing the friction loss for lengths corresponding to smaller ranges of θ and taking the sum for the total loss. Such accuracy is, however, seldom warranted. Under the above assumption, the unit rate of change of the wetted perimeter may be determined by the formula below:

$$x = \frac{2a(\csc\theta_1 - \csc\theta_2)}{L^1} \quad \ldots\ldots (3).$$

The friction head is usually small in short reducing sections, and may be either neglected or computed by the usual approximate methods. In long sections with large changes in velocity a direct and accurate formula is more imperative. Such a formula may be developed by applying the calculus to the Kutter-Chezy formula and making proper substitutions and reductions therein. By assuming c to be a constant, this fundamental equation for determining friction head may be put into the form of equation (4) below for any cross-section of the reducing section at the distance L from its upper end. At this same cross-section, from the relation of velocity, area and discharge, equation (5) follows, and equation (6) may be readily deduced from the definition of hydraulic radius. By substituting the values of v and r from equations (5) and (6)



in equation (4), and integrating between the limits of L^1 and 0 there is obtained equation (7), the desired friction formula.

$$h_f = \frac{1}{c^2}\int_0^{L^1}\frac{v^2\,dL}{r} \quad \ldots\ldots (4)$$

$$v = \frac{q}{a[w + a\cot\theta_1 - (y + 2\tan\phi)L]} \quad \ldots\ldots (5)$$

$$r = \frac{a[w + a\cot\theta_1 - (y + 2\tan\phi)L]}{w + 2a\csc\theta_1 - (x + 2\tan\phi)L} \quad \ldots\ldots (6)$$

$$h_f = \frac{q^2}{2a^3c^2(y + 2\tan\phi)^2}\left[\frac{(w + 2a\csc\theta_1)(y + 2\tan\phi) + [w + a\cot\theta_1 - [w + a\cot\theta_1 - 2(y + 2\tan\phi)L^1](x + 2\tan\phi)}{(y + 2\tan\phi)L^1]^2}\right.$$
$$\left. - \frac{(w + 2a\csc\theta_1)(y + 2\tan\phi) + (w + a\cot\theta_1)(x + 2\tan\phi)}{(w + a\cot\theta_1)^2}\right] \quad \ldots\ldots (7)$$

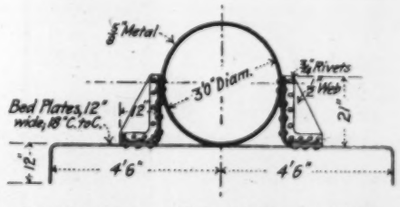
By summing the results of equations (1) and (7) the total head required in the converging reducing section is obtained. Equation (1) is directly applicable to the case of the diverging reducing section, the value of h_v being negative showing that there is a surplus head. In equation (7), in order to compute the loss of head due to friction for a diverging reducing section, the sign of ϕ, x and y should be changed. There will be little, if any, loss of head due to expansion of the canal cross-section, as the change is made gradual by means of the warped surfaces of the reducing section. (See Treatise on Hydraulics, by Mansfield Merriman, p. 174). Therefore, by making an algebraic summary of the heads computed by equations (1) and (7), as above outlined, the excess head in the diverging reducing section may be obtained.

In order to illustrate the use of the foregoing formulas, let it be required to find the drop to provide in a concrete converging reducing section having a length of 100 ft., a bottom width of 15 ft. at the initial and 10 ft. at the terminal end, a uniform water depth of 5 ft., side slopes of 45° at the initial and 90° at the terminal end, warped side surfaces, and a capacity of 500 sec.-ft. Here $v_1 = 5$; $v_2 = 10$; $\theta_1 = 45$; $\theta_2 = 90$; $L^1 = 100$; $q = 500$; $a = 5$; $\tan\phi = 0.025$; $w = 15$; and $c = 120$ (c has been taken as the arithmetical mean between the value of this coefficient at each end of the reducing section based on a value of 0.015 for n and on the slope necessary to produce the required increased velocity head). By substituting these values in the proper equations and making the required reductions, slide rule computations give the following results: $h_v = 1.17$; $y = 0.05$; $x = 0.04$; $h_f = 0.12$. The total drop required in the reducing section, being the sum of h_v and h_f, would, therefore, be about 1.3 ft. It will be noted that the friction head is quite small in the problem solved, as might be expected. Had the length been greater and the change of velocity more considerable, the results and the complicated solution would seem more commensurate.

A UNIQUE RIVETED STEEL PIPE SEWER OUTLET EXTENSION, AT BLACKPOOL, ENGLAND.

In extending the sewer outfalls at Blackpool, England, an interesting detail, shown by the accompanying illustration, was introduced by Mr. J. S. Brodie, M. Inst. C. E., Borough Engineer and Surveyor. The main outfall sewer of Blackpool discharges into the open sea at a point on the water front near the center of the town. The sewage and storm water have heretofore been delivered separately through two 36-in. cast-iron pipes, supported by means of piles and cross timbers. This old outfall terminated at the low-water level of high spring tide, sewage being discharged only on the first four hours of the ebb. The formation of a sand bar near the old outfall made an extension of the old outfall necessary. It was estimated that the 450-ft. extension proposed and subsequently carried out would have cost some $50,000 to $75,000 if it had been of construction similar to the old outfall. By the substitution of riveted-steel pipes, the work was finally completed within the estimate of some $15,000.

The 3-ft. steel extensions were made from ⅝-in. lap-welded pipe delivered in 18-ft. lengths. These lengths were riveted together on shore by



Steel Outfall Sewer, with Bed Plates, at Blackpool, England.

means of ⅝ × 10-in. cover straps, two rows of ¾-in. cup-headed rivets at 3-in. pitch being used. The novel feature of the design was the provision of steel bed-plates, as shown by the illustration. These plates were spaced 18 ft. c. to c., and were designed to prevent the pipe from rolling after being put in place. The addition to the outlets, after the pipes were riveted together on shore, was floated out on high tide, swung into place, and connected up. The method of doing this part of the work is described in some detail in the London "Contract Journal" for Aug. 14, 1907, from which our information is taken.

STRESSES IN LATTICE BARS OF BUILT COLUMNS.

By THOMAS H. JOHNSON,* M. Am. Soc. C. E.

This tentative study assumes that the law of column resistance is correctly represented by Euler's equation and a straight line tangent thereto. (See Transactions Am. Soc. C. E., Vol. XV., pp. 517 et seq.) It is further assumed that inasmuch as Euler's equation is based on *flexure* alone, without *compression*, therefore at the length-ratio at which the straight line becomes tangent to the curve, the resistance to compression becomes zero, and the resistance at that and all greater length-ratios is wholly a resistance to flexure, in conformity with that equation. On the other hand when the length-ratio approaches 0, the flexure disappears and the resistance is wholly to compression. Between these points the resistance is partly to compression and partly to flexure.

The foregoing assumptions will no doubt be questioned, as they have been in the past. But they are based on the study of a large mass of physical tests, which are actual facts; and in all scientific investigations an array of facts is of greater credibility than are theoretical deductions from abstract hypotheses.

Referring now to the diagram herewith: Let $A\ T$ be a tangent to the curve of Euler's equation, at T, and meeting the vertical axis at A, making $O\ A = K$ the allowed unit stress for compression alone. The ordinate $B\ T$ at the point of tangency is given equally by Euler's equation

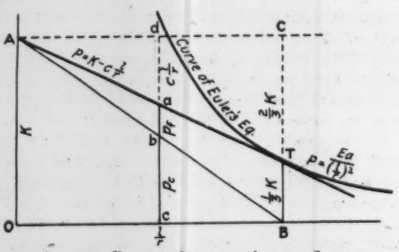
$$P = \frac{E\,a}{(l/r)^2}$$

and by that of the tangent

$$P = K - c\,\frac{l}{r}$$

and, by a special property of this curve, $B\ T$ is always equal to $\frac{1}{3}\ A\ O$, whence $B\ T = \frac{1}{3}\ K$, and $T\ C = \frac{2}{3}\ K$, or $B\ T = \frac{1}{2}\ T\ C$.

Now, as previously stated, the unit stresses represented by ordinates to the curve to the right of B are due wholly to flexure, and be-



tween O and B are due partly to flexure and partly to compression. And here we come upon a point wherein our technical knowledge is deficient. We do not know and have no data from which to determine the law by which the division of the unit stress, as between compression and flexure, varies with increasing values of $\frac{l}{r}$ from O to B.

The portion due to compression diminishes from the value at K at O to zero at B. If the rate of decrease is proportional to $\frac{l}{r}$ then a straight line from A to B will divide all ordinates into parts p_c and p_t representing the unit stresses due respectively to compression and to flexure. If, however, the rate of decrease is proportional to some other function of $\frac{l}{r}$, the line joining A and B will be curved, convex either up or down as the case may be. But in either case the expression for p_c must be of such form that $p_c = K$ when $l/r = o$ and $p_c = o$ when $l/r = O\,B$.

In the absence of definite data, we will assume for our present purpose the first condition, and draw the straight line $A\ B$. Then the ordinate $a\ b\ c$ at $\frac{l}{r}$ is divided by this line into the two parts $a\ b$ and $b\ c$, respectively designated as p_t and p_c. The portion of the unit stress due to flexure, p_t, is readily determined as follows.

In the figure, if we extend the ordinates above the line $A\ T$ to the horizontal line $A\ C$, the portion of any ordinate included between $A\ T$ and $A\ C$ determined by $c\,\frac{l}{r}$, the negative term in the general equation $P = K - c\,\frac{l}{r}$.

Since $B\ T$ is one-half of $T\ C$, therefore $a\ b$ is one-half of $a\ d$, from which

$$p_t = \tfrac{1}{2}\ c\,\frac{l}{r} \qquad \ldots\ldots(1)$$

The unit-stress P is always treated as a mean stress applied over the entire area of the cross-section; but in so far as a portion of it is due to a bending moment, that portion must be considered as applied to the half area on the compression side of the neutral axis. The reduction of the compressive stress on the tension side does not concern us in this study.

The stress p_t, then is the mean compressive stress due to bending; that is to say, the unit-stress at the center of moments of the half section, or at the distance r from the neutral axis. Hence, if t denote the distance from the neutral axis to the extreme fiber, then the extreme fiber stress will be

$$f = \frac{p_t\,t}{r} \qquad \ldots\ldots(2)$$

By substitution of (1) in (2),

$$f = \frac{c\,l\,t}{2\,r^2} \qquad \ldots\ldots(3)$$

To find the shearing force transmitted through the lattice bars we must first find an equivalent transverse load that would produce the same fiber stress f. For this purpose we will take the case of a center load on a supported beam, in which

$$M = \frac{W\,l}{4} = \frac{f\,I}{t} = \frac{f\,A\,r^2}{t}$$

Hence

$$W = \frac{4\,f\,A\,r^2}{l\,t}$$

Substituting for f its value from Eq. (3)

$$W = 2\,c\,A \qquad \ldots\ldots(4)$$

The corresponding shear in the latticing on each side will be

$$S = \tfrac{1}{4}\ W = \frac{c\,A}{2} \qquad \ldots\ldots(5)$$

and the stress in the axial direction of the lattice bars

For angle 45°, $S_1 = 1.4\ S = 0.7\ c\ A$ .....(5a)
For angle 60°, $S_1 = 1.2\ S = 0.6\ c\ A$ .....(5b)

When the general equation is used in the form $P = (K - c\,\frac{l}{r})\ (1 + \phi)$ the last three equations must be written

$$S = c\ (1 + \phi)\ \frac{A}{2} \qquad \ldots\ldots(5_1)$$

for 45°, $S_1 = 0.7\ c\ (1 + \phi)\ A$ .....$(5_1\,a)$
for 60°, $S_1 = 0.6\ c\ (1 + \phi)\ A$ .....$(5_1\,b)$

Example. Given a column in which the area of section is 800 sq. ins., designed by the general equation,

$$P = (12{,}000 - 50\,\frac{l}{r})\ (1 + \phi)$$

also, given $(1 + \phi) = 1.75$ and the angle of lattice bars 45°.

Then we have, from Eq. $(5_1\,a)$, the stress in the axis of the lattice bar

$$S_1 = 0.7 \times 50 \times 1.75 \times 800 = 46{,}600 \text{ lbs.}$$

With these data the proper size of angle bar and number of rivets required in the connections may be readily determined. It is assumed that every bridge designer knows that the individual members of which a column is composed must have a unit stress not greater (i. e., $\frac{l}{r}$ not less) than for the column as a whole.

This study is offered, not as a demonstrated proposition, but as a tentative suggestion in the hope that it may stimulate study and investigation along these lines. In the meantime bridge designers may find it useful. Certainly it is better than a mere guess or unguided judgment.

There are two assumptions in this study which may or may not be confirmed by future research. First, that the portion of the unit stress due to flexure is proportional to the first power of the length-ratio. From analogy with the general equation this appeared to the writer most probable. Second, as to the assumed case of transverse loading. In considering the relative merits of a uniform load and a center load, the latter seemed preferable for the reason that its curve of flexure appeared to be more closely allied to that of the elastic line; besides it leads to a uniform stress in all the lattice bars; whereas the alternate case gives stresses decreasing from the ends to the middle of the length.

*Consulting Engineer, Pennsylvania Lines West of Pittsburg, Pittsburg, Pa.

A NEW LINE OF LIFTING MAGNETS FOR USE WITH CRANES.

The commercial application of electro-magnets for handling iron and steel has not heretofore been extensively developed, although experimental apparatus has been tried at a few steel plants during the past ten years. One such, as used by the Illinois Steel Co., is described in a previous issue.*

Lack of early success, where the apparatus was not found applicable, may largely be attributed

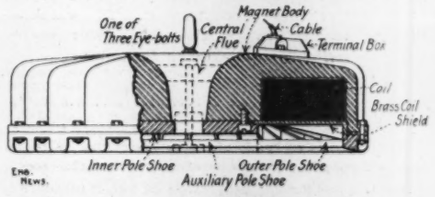


Fig. 1. Sectional View of 50-In. Lifting Magnet; Cutler-Hammer Clutch Co.

to the lack of a line of lifting magnets designed to meet the definite conditions. This lack has been obviated by a few concerns. To meet the demand for commercial lifting magnets, the Cutler-Hammer Clutch Co., Milwaukee, Wis., developed a line of apparatus which has just been placed before the public. The sizes furnished extend from one with a 10-in. face for lifting plates, rails, and small castings, up to one of 52-in. face, designed for handling pig and scrap iron. Wherever large quantities of iron and steel,—particularly in the rough forms,—are to be handled, the great saving of time in the adjusting of hoisting tackle, justifies the additional expense. In foundries and rolling mills the products may be handled before they could possibly be touched by hand, on account of heat retained.

The lifting capacity of such a magnet is rather variable, depending largely on the nature of the material to be transported. With a large mass of iron of such a shape as to afford a fair magnetic contact a 50-in. magnet of the Cutler-Hammer line is claimed to lift some 20,000 lbs., while with contacts and mass conditions unfavorable, for proper magnetic flux paths, the load limit may drop to less than 1,000 lbs. Under common foundry conditions 1,000 to 2,000 lbs. loading would be expected. The capacity of the 10-in. type, shown in Fig. 4, is about 800 to 1,000 lbs. under

*Engineering News, Aug. 19, 1897.

favorable contact conditions. The weight of the members of this line of magnets runs from 75 lbs. in the case of the 10-in size to about 5,000 with the 50-in.

The form of the material handled influences the design. Thus, the larger magnets for handling plates, rails, tubes, etc., are usually of rectangular form and operated in pairs on a balancing-bar, to which the crane hook is attached. For handling pig, scrap and "skulls," the form is circular, and the lifting capacity designed to be larger.

The construction of the large annular type for mill and foundry work is shown in Fig. 1. It consists of a hollow steel casting, of annular shape, in which the magnetizing coil is placed. This coil is built up of copper and asbestos and is insulated from the steel frame by thick sheets of mica. No combustible of any sort is used in the construction. Under test these coils have been raised to 470° F., without injury. In the illustrations it is seen that the larger annular forms have a corrugated frame. There are two advantages of this construction; greater radiating surface is provided and the through bolts shown are protected from injury. The radiation of heat generated by the current is assisted by the central opening, which practically forms a flue through which air may freely circulate. This feature is especially advantageous in handling heated masses of iron.

It has been found that large magnets for handling pig, scrap, etc., are best made concave on their face. Larger numbers of irregular shapes are handled at smaller effort than when the face is plane. Occasionally, however, such magnets are called upon to lift large and heavy pieces with a plane surface. In such a case the concave surface becomes objectionable, due to the intervening air gap. This difficulty is obviated by inserting, in the central opening, an auxiliary pole piece so designed as to extend the inner pole shoe down to the level of the outer pole. This is shown by dotted lines in the section, Fig. 1. By this device both poles are in close contact with the iron to be lifted. The outer pole shoe (see Fig. 1) is easily removable and may be renewed when sufficiently worn to make this advisable. It will be noted that this pole shoe is held on by through-bolts and not by cap screws. Shou'd any bolt not yield to wrenches it is so accessible that the nut may be chiselled off and the shank driven through.

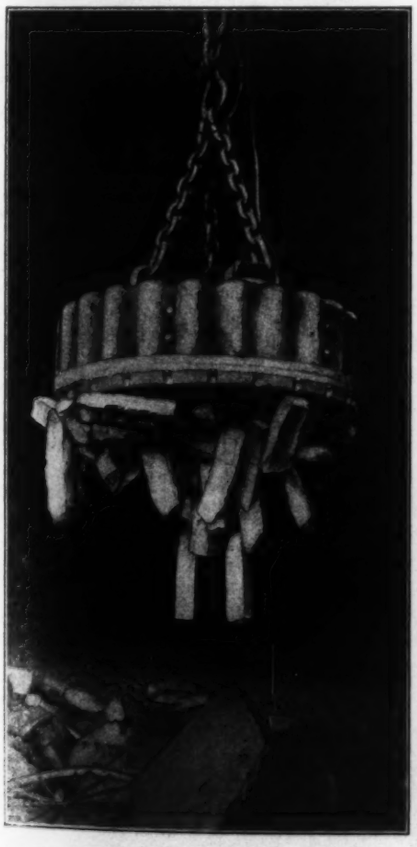


Fig. 3. 50-In. Magnet Lifting Pig Iron.

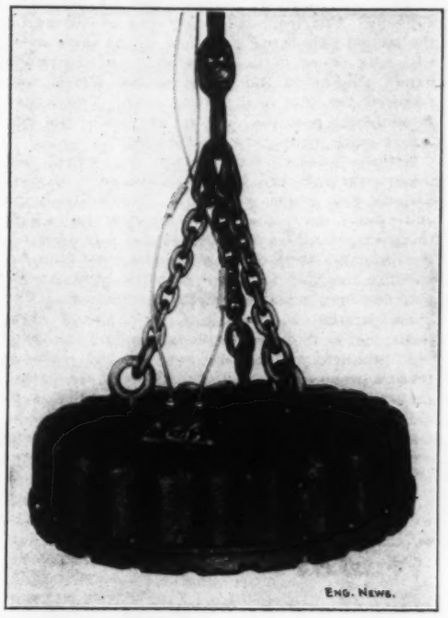


Fig. 2. 50-In. Lifting Magnet, Built for Youngstown Sheet & Tube Co.

At tests conducted at the Donora works of the Carnegie Steel Co., a 52-in. Cutler-Hammer magnet, of the type shown in Figs. 1 and 2, lifted, from the ground, 32 sand cast manganese-iron pigs, a total lift of 2,080 lbs. During the same tests this magnet lifted a 5,500-lb. steel "skull," of uneven surface, covered with slag. In July, 1907, a lifting magnet, of the type described, at competitive tests by the Youngstown Sheet & Tube Co., Youngstown, Ohio, unloaded a steel gondola car, containing 109,350 lbs. of sand-cast pig-iron in 2 hrs. 5 min. The average lift was 785 lbs., and the cost of current for the magnet during the trial was somewhat less than 25 cts. Only one man—the crane-operator—was required. This amount of 785 lbs. may seem small in comparison with the 2,080 lbs. lifted at the Donora plant. It should be borne in mind, however, that at Youngstown the pigs were unloaded from a steel car, containing a large amount of pig-iron. This condition is not as favorable for a heavy lift as when the pig was lifted from the ground.

It is contended in some quarters that there is excessive danger of dropping the load by interruption of the current. It is evident that there must be slightly more danger in this regard than in the use of common tackle. This objection may be largely eliminated by properly safeguarding the magnet circuit, at least, so that interruptions could result only by failure of the primary source or serious trouble on the main circuit from the primary source. Further care would be advisable, on the part of the crane-swinger in warning workmen to stand from under the loaded magnets.

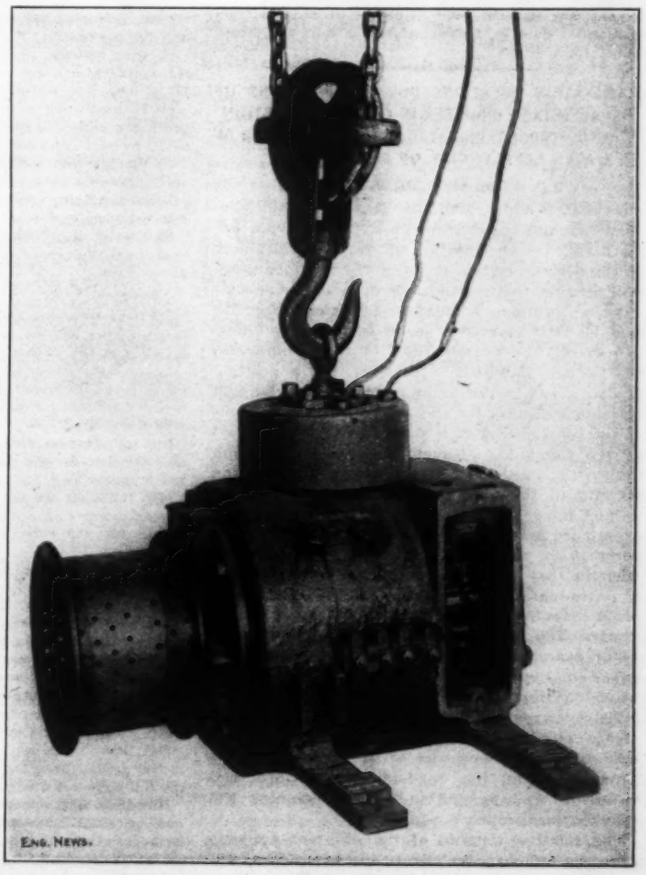


FIG. 4. 10-IN. CUTLER-HAMMER MAGNET, LIFTING CAR LIGHTING GENERATOR WEIGHING 800 LBS.

CORRUGATIONS IN THE ROLLING SURFACE OF RAILS.

A peculiar sort of rail wear which is being experienced generally abroad and to some extent in this country is the development of transverse corrugations on the wearing surface of the head. In this country they are termed corrugated rails, while abroad they are known as "roaring rails" on account of the noise caused by trains passing over them. The trouble is experienced both on ordinary railways and on street railways. On the City & South London Ry. (operated by four-wheel electric locomotives) this wear of the rails has given much trouble. Various reasons have been given for the cause of this form of wear; the vibration of or irregularities in the rolls, defects in the rail supports and ballast, imperfect wheels, the application of brakes, and a general weakness of track in relation to the loads and traffic, have all been urged as direct or contributory causes. The subject has been investigated very carefully by Mr. M. E. Perroud, Engineer of Maintenance of Way of the Northern Ry., of France, and the results of his investigations have been published recently in the "Revue Generale des Chemins de Fer." He arrives at the conclusion that the corrugations are due to the vibrations of the rails as they leave the rolls, and may be either accentuated or smoothed out in the track according to the character of the traffic. With trains moving at very high speeds the wheels strike the faces of the ridges a succession of sharp blows which cause very hard and highly polished spots. He recommends that such rails should have the rolling surface finished or faced in some way before

being laid in the track. On street railways in England the use of grinding machines to smooth off corrugated rails (in the track) is becoming quite general, as noted in our issue of April 11, 1907. The conclusions presented by Mr. Perroud, based upon his investigations are as follows:

1. Rails usually have a slightly undulating surface from the time they are laid on the track.

2. These undulations are due to the vibration of the rail as it leaves the rolls at the mills.

3. These undulations may increase or diminish in accordance with the hardness of the steel, the loads carried, the speed of trains, and the local conditions, as to oxydization. On rails of hard steel in tracks carrying trains at high speed, the corrugations are very sharp and persistent.

4. The deep corrugations in rails of hard steel are independent of the type and length of rail, the character of the track, and the material of the ballast and roadbed.

5. On hard steel rails in service, the depth of the corrugations may be as great as 0.0118-in., but the average is about 0.0078-in. It depends upon the amount of traic and the speed of trains.

6. The peaks of the corrugations are 1.18 to 1.57 ins. c. to c., but the distance varies in rails from different mills.

7. After leaving each peak, the wheels having high speed of translation are practically free from the rail and strike the rising face of the next peak.

8. These blows hammer the ridges, causing polished spots of great brilliancy, and compressing the metal so as to make it extremely hard at these spots.

9. The blows from wheels traveling at lower speed, as well as the effect of corrosion, have more action upon the hollows than upon hard ridges and therefore tend to deepen the corrugations.

10. Blows from wheels at intervals of about 1-1000-sec. cause hard ridges about 1.18 ins. apart, and trains running at 60 miles an hour produce a characteristic sound.

11. The alternating light and heavy pressures of the wheels upon the rails, cause deflections of the track which, being repeated at short intervals, tend to disturb the ballast tamped under the ties. Thus tracks laid with hard steel rails having marked corrugations require more maintenance than those in which the rolling surface of the rails approaches more nearly to an ideal plane surface.

12. It appears possible to prevent the difficulty, noise and expense, by judiciously facing the rails (by grinding, planing or rolling) so as to remove the incipient roughnesses. For economy, and to ensure permanance, this treatment should be applied before the rails are placed in the track.

EXAMINATION QUESTIONS FOR THE POSITIONS OF "ASSISTANT ENGINEER IN CHARGE OF SECTION" AND "TOPOGRAPHICAL DRAFTSMAN," BOARD OF WATER SUPPLY, CITY OF NEW YORK.

By SUBSCRIBER.

On May 8 and 9, 1907, the Municipal Civil Service Commission of New York City held an examination for the position of Assistant Engineer in Charge of Section, it being the first examination held for this position. It departed somewhat from the ordinary custom of the Commission, inasmuch as it covered a period of two days while, in general, the examinations of the Commission are for one day only. However, for a position of that grade, requiring as it does knowledge and training sufficient for the successful carrying out of the construction of a work of such importance as the Catskill Aqueduct, one can hardly find fault with the length of the examination or the severity of the questions asked.

Candidates were required to have ten years' experience and to show at least two years of experience in charge of work or in a position wherein they had some authority or responsibility. Graduation from a technical school of recognized standing was counted as two years' experience. The minimum age limit was 28 years.

The examination proper comprised a technical paper, in two parts, a mathematical paper, and a report. In addition to these there was an experience paper which was forwarded to the candidate with his card of admission to the examination. This paper was to be filled out in the handwriting of the candidate, subscribed and sworn to, and handed in before beginning the examination proper.

The relative weights of the different subjects were as follows: Technical, 40; experience, 30; report, 20; mathematics, 10. To pass a percentage of 75 was required on the technical paper, an average of 70 on all, and at least 20 on each subject.

A Topographical Draftsman examination was held on May 29, 1907. The papers of the examination proper were in technical and mathematical subjects. The technical paper was in two parts, the second part being combined on the same sheet with the paper in mathematics. An experience paper, similar to the one described above, was required for this examination also. The writer is unable to give the relative weights of the different subjects in this examination.

Within the last few months, the writer has taken several of the Commission's examinations for Board of Water Supply positions and must say that the manner in which these examinations were conducted was very refreshing indeed to one who knows something regarding the way in which so-called civil service examinations are sometimes carried out. The examinations comprised good, fair, honest questions, just such as would likely come up in everyday practice. Except in possibly one instance there were no catch questions. Some of the mathematics, however, contained such numbers as to make the operations long and tedious. So far as the writer's experience goes the examinations were conducted in an entirely open and aboveboard manner.

The writer is of the opinion that the questions as given hereafter, while not being in the exact language submitted to the candidates, are sufficiently near it to give quite an accurate idea of the scope of the examination:

EXAMINATION FOR ASSISTANT ENGINEER IN CHARGE OF SECTION.

First Day.

Technical Paper. Time, about 10.30 a. m. to 1.30 p. m.

(1) A high masonry dam is to be founded on rock. Give a detailed statement of the examination and preparation of site that should be made up to the time of laying the first stone.

(2) Give the methods that should be pursued and the precautions to be observed in building an embankment for an aqueduct.

(3) Give a paragraph of a specification for earth excavation, embodying method of measurement and classification for payment.

(4) Give sketches showing approximate size of timbers for a cofferdam about 10 × 18 ft. in size, in water say 17 ft. deep, the bottom being gravel.

(5) In excavating a tunnel shaft describe the method you would pursue to make the excavation with least difficulty, in case quicksand were encountered.

(6) Give sketches and description of a culvert about 35 sq. ft. in cross-section, through a high embankment for an aqueduct, most suitable, in your opinion, to ensure safety against washouts, etc.

(7) Give a description of means and instruments that experience has proved to be best for depositing concrete under water.

Technical Paper. Time, 1.30 to 4 p. m.

(8) If necessary to join new concrete of an aqueduct to concrete that has set for quite a long time give a description of the precautions that should be observed.

(9) In using asphalt on a street, or say for waterproofing a large reservoir, what precautions are absolutely essential in order to make the asphalt form a good union with the cement?

(10) (a) What material (or materials) are used for puddle; (b) describe the manner of placing and state how wet it should be.

(11) (a) What are the requirements for steel for reinforcing concrete as to the condition it should be in when it goes into the work; (b) as to placing; (c) as to position.

(12) (a) Describe in detail how you would go about an examination of rock on the site of the line of the aqueduct for determining its fitness for use in permanent work. (b) What would be cause for rejecting sand and gravel to be used in concrete where impermeability is desired? (c) What are the requirements for sand and gravel where impervious concrete is desired?

(13) (a) Sketch and describe the two principal methods of timbering tunnels; (b) state which is to be preferred, where applicable and why.

(14) Describe the method employed in building a tunnel in bad ground from the time the heading is started until the full section is attained.

(15) Make out a monthly estimate giving approximate unit prices and including all kinds of excavating, embankment, tunnelling, road making, fencing, etc.

Second Day.

Mathematical Paper. Time, about 10 a. m. to 1.30 p. m.

(1) A dam is 60 ft. high, 8 ft. wide on top, 36 ft. wide on the bottom, has face on water side vertical, [illegible] built of masonry weighing 150 lbs. per cu. ft. The [illegible] level is 3 ft. below the top. Determine where the [illegible] sultant of forces will pierce the base.

(2) A gate is 3½ ft. high by 3 ft. wide, with the [illegible] ft. below the water surface. What force will be nec[illegible] to start it if the coefficient of friction of quiesce[illegible] say, 40%?

(3) A tank 20 ft. in diameter and 15 ft. high is f[illegible] water. The hoops are spaced 18 ins. c. to c., the [illegible] one being 6 ins. above the bottom. Determine the [illegible] section of the lower and the next to the lower hoop[illegible] posing 12,000 lbs. stress per sq. in. is allowed o[illegible] metal.

(4) The perimeter of a hexagon is 800 ft. What [illegible] area?

(5) The arcs of two concentric circles are 325 and [illegible] ft. in length, the central angle in each case being [illegible] 57′ 13″. What is the area included between the arcs [illegible] the radii?

Report. Time, 1.30 to 4 p. m.

Write a report, properly addressed, of about four p[illegible] (more may be written if desired) respecting work of [illegible] kind on which you have held a responsible position. State clearly what the position was, how long you held it, and give details of construction, incidents, experiences, impressions of the work, and any data that will help to show your skill as an engineer, your ability as an executive officer, and how resourceful you are in case of sudden emergencies.

EXAMINATION FOR TOPOGRAPHICAL DRAFTSMAN.

Technical Paper, Sheet 1. Time, about 10.20 a. m. to 12.30 p. m.

(1) Show in 1-in. squares draftsman's standard methods of showing the following in cross-section: (a) concrete; (b) sand; (c) earth at surface; (d) water; (e) hard rock.

(2) Describe and show the signs for the following: (a) frame house and stone house; (b) property, township, and county lines; (c) highway and railway bridges.

(3) What are the symbols for triangulation, stadia, traverse station, and a bench mark?

(4) (a) What is the principle of the metric system? (b) What is the value in feet of a meter? (c) Approximately, what is the value in miles of a kilometer?

(5) What are the several cases in the solution of a triangle, and how are there sometimes two solutions?

(6) Give clearly, step by step, how you would examine the notes of a boundary survey for an area and determine if it closed, and if not how to distribute the error.

(7) (a) What is a traverse table and how is it used? (b) What is a logarithm, and to what uses are tables of logarithms put?

(8) How should a triangulation survey be plotted?

(9) Explain how the configurations of the earth are shown on a topographical map.

(10) Define and explain the difference between Normal, Perpendicular, Vertical, a Right Angle, and Plumb.

Technical Paper—Sheet 2—and Mathematics. Time, 12.30 to 4 p. m.

Technical Paper, Sheet 2.

11), (12), (13) In a rectangular space about 3 × 6 in. in size show a portion of a topographical map giving cleared and cultivated land, cliffs, a stream, oak trees, orchard, country road, farm house and barn.

(14), (15) Lay out and show in four styles of lettering and fill in at least three letters of each style, the following title: City of New York, Board of Water Supply. Standard Topographical Symbols. Scale, 100 ft. to 1 in. May 29, 1907.

Mathematics.

(1) The side of an equilateral triangle is 363 ft. What is the distance from an apex to the center of the opposite side?

(2) The sides of a triangle are 10, 20 and 30 ft. What is the sin. of the angle A (the angle between the sides 10 and 20)?

(3) An error of one minute is made in measuring an angle. What should be the offset for the true line at a distance of 1,000 ft.?

(4) What is the difference in feet and decimals between 3 ft. 9½ ins. and 5.64 ft.?

(5) A street rises on a grade of 89.2 ft. per mile to a point A, from which point it falls at the rate of 118.8 ft. per mile. At a point 492 ft. before A is reached the elevation is 192.07. What is the elevation of a point 1,200 ft. beyond A?

Refering to the list of questions given on page 195 of Engineering News for Aug. 22, 1907, and to the statement that 14 of the 15 questions propounded are given, as a matter of interest and in order to make the list complete, the writer believes the omitted question to be somewhat as follows: "Give the usual specification requirements for slopes for excavation and embankment for rock, gravelly loam, and clay."

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The investigation of the Quebec Bridge collapse by the Government Commission of Engineers, Messrs. Holgate, Kerry and Galbraith, is proceeding steadily. During the past week additional evidence has been obtained showing that in the compression chords of both anchor arm and cantilever arm certain members were in an unsatisfactory condition for some weeks before the final catastrophe. Such evidence is difficult to procure, as some of those who knew most about the state of affairs went down with the bridge. It is evidence based on recollection, given without the help of notes, measurements or photographs, and is for that reason uncertain, and the truth must be sought by careful sifting. But the inquiry is more and more fully establishing the fact that the bottom chords of the Quebec Bridge were far from ideal, not when erected but afterward, when they began to take stress in increasing amounts. Their several ribs began to exhibit slight bends, they were in a special state of lateral constraint at the field splices, and in at least two or three cases a chord section was obviously out of straight.

The question of supreme interest to the engineer is not the primary one of what member failed first, or the legal one of why work was continued regardless of these evidences of distress, but why the chords were distressed at all, why they behaved as they did. As we remarked immediately after the collapse, the fact must be recognized that the design of these great struts was not at all abnormal, except that they were of unprecedented size; and their unit-stresses were not high, at the time of the collapse. We have made calculations, based on an estimate of the difference between the loading on Aug. 29 and the full dead-load, of the stresses in the anchor-arm chords; they indicate that the highest compression in these chords was not far from 12,000 lbs. per sq. in. The question then is: under what conditions can a short, thick strut, loaded to only 12,000 lbs. per sq. in., exhibit clear signs of straining out of its designed shape? The nature of the shop processes used in building and finishing these chords, and of the procedure followed in riveting up the field-splices, have an important bearing on this problem. The investigation now in progress may be expected to shed additional light on these matters.

A few years ago at a general faculty meeting of one of the smaller, but well known, universities an elderly professor of Latin passed a remark in which he dubbed the chemical students as "pot-boilers." This light speech brought the senior professor of chemistry to his feet with the vigorous expression that, "If the reverend gentleman knew all that lay behind the boiling of a pot, he would be nearer his Creator than ever before."

The suavity and confidence with which the text books of a few generations past have explained the apparently simple phenomena of daily life, such as the rusting of iron, brings such incidents as noted, strongly to mind, not because of a superficial quality of humor but because of a compelling note of truth in them. The electrolytic theory of rusting is far from the simple little chemical explanations taught a generation ago. It involves the deepest physical and chemical science, and perhaps approaches that border line of fact and fancy over which the most eminent scientists of the times are disputing. The main factor of the theory is the ion—one of the most minute imaginary divisions of matter in present physical concepts. The electrolytic theory must be admitted, however, as the most adequate, satisfying and logical one yet developed. It is broad and comprehensive, including the former theories as specific or special cases of a more general process.

The practical utility of the new explanation of the corrosion of iron will be measured by the assistance it may give in the prevention of rusting. From the older theories it was seen that air and moisture were to be kept away from the iron or steel and protective coverings have been developed with the idea as fundamental. Paints that were thoroughly or permanently non-porous have not appeared and iron structures still rust in spite of the best of care. It would seem possible that the additional knowledge gained from the experiments carried on in developing the new theory, could lead to the invention of some protective covering that would inhibit the electrolytic action. Possibly the agitation may develop a truly non-porous protection which is practical.

Why does iron corrode so much more easily under common conditions, than metals like copper, lead and zinc? This has been a common question with no adequate answer. The electrolytic theory leads to an answer, at least plausible, which is presented at face value only. In Ostwald's table, as shown elsewhere in this issue, iron is shown to be close to the critical point of change in ionic action, while the other metals named are distant. This position of iron indicates that while there is a tendency for metalic ions to pass from the metal to the solution in which it may be immersed, the common physical and chemical irregularities are probably great enough to reverse in certain areas the normal tendency, causing more continuous electrolysis and rendering electrical equilibrium more difficult. In the case of the other metals, in samples that have resisted corrosion, it may safely be assumed, first that the physical and chemical heterogenity is less marked than with iron. These metals being farther removed from the critical point of change in ionic action, the influence of such irregularities in the metal may easily be conceived as unable to reverse the normal tendency for the exchange of metallic ions between metal and solution. The potential difference between the metal and the solution is then always undirectional though slightly varying at different parts of the metal-body. The light resultant currents and the very slight electrolysis would tend to equalize the potentials, establish electrical equilibrium and limit further corrosion.

The hooped column has had a more rapid growth of popularity than perhaps any other device in reinforced-concrete construction. Concrete columns have awkwardly large cross-sections when designed by conservative unit-stresses, and building designers strongly dislike to sacrifice more than the absolute minimum of floor space to structural elements. The hooped column, capable of carrying much higher unit-stresses, afforded a means for greatly reducing the sizes of concrete columns. Its popularity is based entirely on this fact, we believe, rather than on considerations of economy.

The high unit-stresses which designers have applied to hooped columns were based on tests in which such columns were loaded to the failure point. The argument tacitly used was that a stress of 1,500 lbs. per sq. in., applied to a column which would not fail below 6,000 lbs. per sq. in., gave a safety-factor of four, just as large a factor as obtained in a plain concrete column loaded to 500 lbs. per sq. in. when its ultimate strength is 2,000 lbs. per sq. in. But there is far more to the question than such a simple comparison implies. Prof. Talbot's tests, recorded on another page of this issue, throw a more searching light on the behavior of hooped columns. It appears that such columns undergo curious changes in their elastic behavior in the course of a test to destruction. They seem to have a sort of elastic limit, reached even before the loading at which a plain concrete column would fail. It will be a matter for serious consideration whether this point of elastic change must not be taken as the governing strength in determining the allowable unit loads on hooped columns.

Although Prof. Talbot's paper is incomplete, in that it says nothing of the set, or permanent distortion, yet one may conclude from his figures and curves that very large permanent deformations occur in that stage of the loading where the hooping is active. Such deformations may not be very objectionable in some cases, but in many others they would be fatal to the continued usefulness of the column. For example, if one column in a tall warehouse or office building were to suffer a permanent compression due to a temporary overload, then the beams connected to this column would remain in a state of strain of unknown character and therefore dangerous. Such a condition could not be contemplated with indifference by the designer. He would probably prefer, in designing, to keep the unit-stresses in the column so low that the condition described could never arise during the life of the structure.

The case is well summarized in the observation that up to a certain load the hooped column behaves just the same as a plain concrete column, while at higher loads the hooping seems to govern the elastic behavior. Now, this critical load is stated to be roughly the same as the failure load of an unreinforced column. One is tempted to jump at the conclusion that the hooping is inactive until the concrete itself virtually fails. So radical a conclusion is not justified by the experimental data yet to hand. But the great change in modulus of elasticity at this point does undoubtedly indicate a great change in the nature of the compressive resistance. The probabilities are that this change is not reversible, and for this reason must be avoided even as a remote chance in most classes of work. It would follow from this that the unit-stresses on hooped columns cannot be raised so limitlessly as some designers have thought permissible. The matter of fixing the limits of permissible stress will prove a delicate one; fuller experimental data must be had, however, before this can be done and before broad conclusions can be drawn.

A SIMPLE WAY OF MAKING WATERPROOF CONCRETE.

That ordinary concrete can be rendered practically impervious to water and at the same time increased in strength by merely adding a small amount of ordinary clay to the cement is the very surprising and very important statement which is set before the engineering profession, in this issue of Engineering News, by a high authority.

As our readers know, the proper waterproofing of concrete is one of the most important subjects now occupying the attention of engineers. Ordinary concrete is exceedingly porous. In-

stances are on record, we believe, where moisture has traveled for hundreds of feet through solid concrete. There are cases where engineers are going to great expense—hundreds of thousands of dollars in fact—to waterproof the surface of concrete structures. There are other cases without number where we resignedly allow water to percolate through our concrete structures because we cannot help ourselves. In making reinforced concrete for example, untold tons of iron and steel are being buried in concrete, where water is certain to work through and come in contact with the surface of the iron. We are relying on the chemical action of the cement, not its mechanical action, to protect the iron from corrosion; and not a few members of the profession are nervous lest at some time or other, under some circumstances or other, corrosion may after all take place.

And if the steel in reinforced concrete does rust, it means not merely the weakening of the steel; but the actual disruption of the concrete itself, which is a still more serious matter. When iron or steel rusts, as everyone knows, it increases largely in volume. If a rod imbedded in concrete forms rust of any considerable amount, the concrete will be burst open by a force as resistless as that of freezing water.

If, then, concrete can be made which will shut out water instead of absorbing it like a sponge, the fact is of momentous importance to every structural engineer, to every hydraulic engineer, to the builder of concrete sewers, the engineer responsible for street paving—in fact all through the length and breadth of the profession, the possibility even, of impervious concrete is of deep significance.

There have been numerous investigations in recent years in connection with the waterproofing of concrete. These investigations have generally taken the form of tests of waterproofing processes and methods now on the market. There may be said to be two theories in vogue respecting waterproofing. One theory is that an impermeable exterior coating should be applied to prevent water from reaching the main body of the concrete. The other theory is that an interior mixture should be provided to fill up the voids in the mass and so mechanically prevent the passage of the water. Some of the processes deduced from each of these theories have been fairly successful but none so far developed have wholly solved the large problem involved.

The exterior coating waterproofings are as a rule very expensive, incapable of being placed in water, where a large portion of waterproofing work must be done, and are of too fragile a nature to be the sole defense of the structure against percolation. The high efficiency of the interior mixtures has had much experimental verification and as much denial; opinions as to their value differ with the number of engineers using them. One of the greatest arguments against them is that as a rule they materially reduce the strength of the concrete.

With a full knowledge of the unsettled state of waterproofing practise, the Board of Water Supply of New York City started this year to investigate the subject in a more thorough manner than ever before. The immense amount of concrete work to be laid on the new water supply system of the city required that the best efforts be put forth to render the pipe-line siphons and dams as nearly impervious as possible, and that this should be done with the least possible expense.

These tests have been carried on during the last summer and have been marked by a preliminary study of the subject not before undertaken in similar investigations. In the present issue of Engineering News we publish a paper presented by Mr. R. H. Gaines, Chemist to the Board, to the American Society of Civil Engineers, which not only offers most important practical information regarding the waterproofing of concrete, but presents a unique theory concerning the setting of cement.

As most of our readers are aware, knowledge regarding the nature of the process of cement setting is at present indefinite. The old idea of a purely chemical reaction is being rejected and as yet no entirely satisfactory substitute has been advanced, but the one proposed by Mr. Gaines is at least novel and in line with the many recent explanations of the phenomena of nature by the laws of the newly developed physical chemistry, a subject that the average engineer only knows as a study of his college days or not at all if those days are more than a dozen years behind him. However, as the results obtained from this scientific theorizing are decidedly simple and entirely capable of being understood, a study of the method of deduction should be well worth while.

Under this idea, the electrically charged ions of water not only materially aid the chemical combinations of the elements in the cement but also tend to form a glue-like film around the particles, joining them together and forming a dense and hard mass. By the substitution of a dilute solution of some electrolyte for the water, the more numerous ions of the electrolyte will cause the same binding action to a higher degree and the resulting product will be more closely bound together, that is more dense and therefore stronger and more impervious. Also by the introduction into the mass of certain substances known as colloids, which have the power of aiding the electrolytic action and at the same time of retarding the too rapid hydration of the calcium ingredients of the cement the setting process can be carried to a higher degree of density and solidity.

With these theories, the experimenters set to work to mix a cement mortar with a small percentage of electrolyte added, another with the addition of a colloid, in this case pulverized clay, and a third with a combination of the two. Tests were made for strength in 7, 28 and 90 days and for permeability in 28 days under 40 and 80 lbs. pressure and in every case the treated mixture showed a strength far above those not treated and an imperviousness to water practically perfect.

The idea of introducing an electrolyte to increase the strength and watertightness of a concrete is new, so far as we know, although some of the patented mixtures on the market may owe their valuable properties to this quality. On the other hand, the presence of free pulverized clay in a concrete mixture has long been known to add to its strength, though no claim has been made as to its waterproofing qualities. We have published tests showing the additional strength produced by a clay addition in our issues of Nov. 19, 1903, p. 443; Feb. 2, 1905, p. 127; and Aug. 10, 1905, p. 140, but in none of these tests did the experimenters pretend to account for the increased values.

Up to this time, in fact, the engineering profession has, very properly held a most conservative attitude respecting the use of clay in making concrete. It has been felt that while the engineer who is by geographical location compelled to use a sand carrying more or less clay, might perhaps feel safe in so doing, wherever clean sand is obtainable, it is safer to stick to the time-honored specification of "clean, sharp sand" and insist on its fulfilment. The practical reasons for this were well set forth in a symposium in our issue of June 6 last.

So long as the only gain to be expected by the addition of clay was a certain percentage in tensile strength, it was rightly felt that the gain did not compensate for possible risk of deterioration with the lapse of time and of lack of uniformity in composition through failure to perfectly incorporate the clay and cement.

But if the addition of clay makes the cement impervious after setting as well as adds to its strength, the case is very different. A hundred pounds more or less of unit strength in a long time test of good Portland cement is of no very great importance, but waterproof qualities, particularly if they can be secured at trifling cost are of enormous value. We make no guarantee, of course, as to the reliability of Mr. Gaines' conclusions. Naturally a matter of such importance will have to be subjected to repeated trial by many experimenters before the claimed results will be generally accepted by the profession. The simplicity of the process, however, and its practical importance makes it certain that the influence of clay mixtures on cement will be thoroughly investigated in every cement laboratory.

Such investigation is in fact necessary before practical use can be made of Mr. Gaines' discovery. For example, clay is a substance of great physical and chemical variation; different clays may possibly give very different results. It is even possible that clays containing some constituents—iron oxide for example—might eventually give trouble from discoloration, or even lead to disintegration when other clays gave perfect satisfaction. We need to know what clays or what classes of clays may be safely used.

Again, if clay is to be used for waterproofing, how shall it be added? The practical risk in using a sand carrying particles of clay are evident enough. It would seem clear that to secure best and safest results the clay should be first powdered and then thoroughly incorporated with the cement. The difficulty of the average cement user in carrying out this process are evident enough, and if cement is to be "clayed" to make it waterproof, the cement making plant appears to be the place where the work should be done.

There is, perhaps, a danger at this point that some cement manufacturers may engage in "claying" without the customer's knowledge or consent. If a cement maker can add 5% of clay to his product while passing through the tube mill, he can practically reduce the cost per barrel of his output by nearly the same percentage, since the clay will cost him hardly anything; and the resulting cement may be expected to show higher tests than the pure article. Those who draft specifications for the purchase and testing of cement may well investigate this point, for the cement maker who furnishes pure ground clinker should not be compelled to compete with the maker who sells cement mixed with clay. Further, there would be danger that the purchaser of the "clayed" cement might wish to make a further admixture of clay, or use it with a clayey sand.

In the above review of Mr. Gaines' experiments and the practical results likely to grow out of them, we have thus far alluded only to the use of clay in admixture with the cement. The reader will note, however, that experiments were made with alum added to the cement, as well as with clay, and also with a mixture of the two. The alum acted much like the clay in increasing the strength of the cement and rendering it impervious; but the clay gave on the whole considerably better results, both in the short time test and in the tests of the briquettes three months old. As the clay is much the cheaper and more readily obtained, attention may well be concentrated on that by those who would make further test of the remarkable proposition brought before the profession by Mr. Gaines.

LETTERS TO THE EDITOR.

The Deflection of a Beam of Variable Section.

Sir: Could you grant me a little space in the columns of your valuable paper for a problem, the solution of which by some of your numerous readers would be greatly appreciated by me. The problem is to find the deflection at the free end of a cantilever beam loaded in the manner indicated below, taking into account variable moment of inertia.

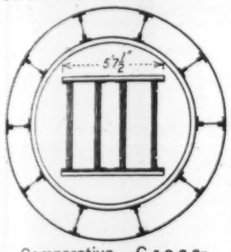
The beam in question is a cantilever beam or bracket, 42 ft. 6 ins. long, 5.77 ft. deep at the inner end and 2.5 ft. deep at the outer end. It is composed of a ⅝-in. web, four 6 × 6 × ⅝-in. flange-angles, a bottom flange-plate 14 × 9-16 ins. by 35 ft. long, a full-length top flange-plate 14 × 9-16 ins. by 42 ft. 6 ins. long, and two more top flange-plates 14 × 9-16 ins., one 11 ft. 3 ins. long, the other 5 ft. 10 ins. long. All flange-plates, of course, begin from the inner or deep end of the beam. The depths at the ends are measured from back of bottom flange-angles to top of full-length upper flange-plate. The beam is loaded with four concentrated loads of 17,000 lbs. each, distant 11.25 ft., 16.75 ft., 26.25 ft. and 31.75 ft. from the inner end.

R. E. L.

Yonkers, N. Y., Sept. 12, 1907.

Compression Members in the Quebec and the Forth Bridges.

Sir: Being greatly interested in the Quebec Bridge disaster, I desire to express my appreciation of the thorough manner in which you have investigated and published account of the collapse. I was in Quebec on Sunday, the 25th ult., and had arranged to inspect the bridge, but rain prevented my doing so. However, I passed the bridge at seven o'clock in the evening on steamboat en route to Montreal. One could not help noting the slim appearance of all the members, comparing it with the only other bridge in the world of the same type and approximately the same dimensions, the Forth Bridge in Scotland, which I inspected when it was in about the same state of completion that the Quebec Bridge was before the collapse. I again inspected the Forth Bridge in 1903, after fourteen years of service. I spent some hours on it, and noted carefully its freedom from vibration, even when express trains were run over it at high speed. I enclose herewith a sketch showing the comparative size of chord member 9 of the Quebec Bridge and the similarly located chord of the Forth Bridge. One cannot help noting the immeasurably superior strength and character of section of the latter, considering that also the Forth Bridge has a less span and greater depth of truss, and that the members of the Forth Bridge are stayed at closer intervals than the Quebec Bridge, although the panel sections are larger and, I think, there is an intermediate stay. While it is more difficult to make satisfactory end connections to a round compression member, the fact that the designers of the Forth Bridge accomplished it leaves nothing further to say on that point.



Comparative Cross-Sections of Bottom Chord of Forth Bridge and Quebec Bridge, at a Point Near Main Pier.

I am looking forward to your further report and discussion on the matter with deepest interest.

Very truly yours,

Frank E. Kirby.

Stevens Bldg., Detroit, Mich., Sept. 14, 1907.

Ultimate Deflections of Columns in Full-Size Tests.

Sir: Under separate cover, I am sending you prints from tracings of some reports of compression tests. These are representative of a series of tests made while I was Inspector of Bridges for the Pennsylvania Lines West of Pittsburg, from which we learned a number of things which have been utilized in the detailing of our work. It was not for this reason, however, that I thought of sending them at this time, but for the purpose of calling your attention to the fact that the lateral deflection does not exceed 0.2-in. within the elastic value of the columns (this is true of all tests made), and in a number of tests this deflection was not exceeded under the maximum load, so that you may understand why I think you would be justified, in writing of the disaster at Quebec, in saying that the failure occurred by reason of the crippling of the bottom chord section A9L. [The columns tested were 14 to 25 ft. long between pin centers, and had sectional areas from 14 to 32 sq. ins. The ultimate load of the largest ran well over one million pounds.—Ed.]

If the deflection noted in that section on Monday prior to the collapse was a segment of a parabola (which is extremely improbable unless the sections in the two adjacent panels were bent outward), it would have been in excess, length considered, of any result I ever noted, and would have spelled "failure" so certainly to me that, had I been on the work, my best efforts would have been immediately exerted to get everybody off the structure.

Yours truly,

C. P. Buchanan.

Pittsburg, Pa., Sept. 13, 1907.

Column Strength and the Quebec Bridge Failure.

Sir: It was with intense interest that I read your reports and comments on the terrible disaster at Quebec, and I must say that your investigations as to the cause of the failure of the Quebec bridge seem to me most thorough and convincing. Your article in the last number of Engineering News, Sept. 12, p. 284, however, in which, after many excellent remarks, you speak about our deficient knowledge of the strength of large columns, conveys the idea—to some extent at least—that the said failure was something inevitable and excusable, or directly traceable to this deficient knowledge. It seems to me that this view is too extreme, and that our present knowledge of the strength of columns is and was sufficiently advanced to design and construct safe and economic compression members for a big structure like the Quebec bridge.

The Quebec failure, to my mind, must not be attributed to the defects of our standard compression formulas, but to the manner in which these formulas were interpreted and followed in designing the chord members of the bridge.

The formulas prescribed for the compression chord were, according to your article, "The Fall of the Quebec Bridge," p. 287:

$$p = 12{,}000\left(1 + \frac{\text{minimum stress}}{\text{maximum stress}}\right) \quad \ldots\ldots\ldots(1)$$

when l is less than $50\ r$

and for main posts:

$$p = \left(12{,}000 - 50\,\frac{l}{r}\right)\left(1 + \frac{\text{minimum stress}}{\text{maximum stress}}\right) \quad \ldots(2)$$

This second formula, I presume, had to be applied also for chord pieces, if l should be found larger than $50\ r$.

Now let us see what values we find for r for the chord piece $A\,9\,L$. The moment of inertia I_h around a horizontal axis is about 208,000, in inches, hence

$$r = \sqrt{\frac{I}{A}} = \sqrt{\frac{208000}{781}} = 16.3$$

As the length l of $A\,9\,L$ centers is 57 ft., or 684 ins., l is smaller than $50 \times r$ and the first formula could be used. We must, however, also consider the moment I_v of the cross-section around its vertical axis.

To find the value for I_v we have first to ascertain to what extent the four plate-girders or ribs forming the chord piece, which are connected to each other by light angle lacing, can be considered as one body. In doing so we must remember that the introduction of the moment of inertia or the radius of gyration in the column formula indicates that we have to do with a problem in which bending forces play an important part, similar to the problem which we meet in calculating and dimensioning loaded beams. We know that in figuring beams an integral moment of inertia can only be calculated for such parts of the cross-section (chords, webs) which are actually and properly connected with each other, so that the connections can withstand the shear between the several parts, particularly between the web and chords, when the beam is exposed to bending. From the theory of beams we know that the amount of shear S, taking place between two cross-sections of the beam for which the bending moments are M_1 and M_2, is:

$$S = \frac{M_1 - M_2}{h} \quad \ldots\ldots\ldots(3)$$

where h represents the average effective depth of the girder between the two cross-sections. Further we know the relation between the bending moment M and the moment of inertia I of a beam. It is:

$$M = \frac{2\,I\,K}{H} \quad \ldots\ldots\ldots(4)$$

where K represents the maximum fiber-stress and H the extreme depth of the girder. We can now, by means of the formulas (3) and (4) calculate for a chord piece the average shear per lineal foot which must be provided for, if the moment of inertia of the chord piece is known, and if the same shall be made effective within maximum fiber stresses.

We substitute in (4), $I = 208{,}000$, $H = 54$ ins., and $K = 16{,}000$ lbs. per sq. in., and obtain the value of M:

$$M = \frac{2 \times 208{,}000 \times 16{,}000}{54} = 123{,}200{,}000 \text{ in-lbs.}$$

If the chord piece $A\,9\,L$ would have been used as a free span, this maximum moment M could have taken place in the center of $A\,9\,L$, whereas the bending moments at the ends of $A\,9\,L$ might have been equal to zero. (This assumption agrees best with Euler's formula II.)

We can make use now of formula (3) substituting for

$M_1 = 123{,}200{,}000$ in.-lbs.

$M_2 = 0$.

$h = 40$ ins.

and we find for the shear S from the end to the center of the chord piece, that is for $\frac{57}{2} = 28.5$ lin. ft.,

$$S = \frac{123{,}200{,}000 - 0}{40} = 3{,}080{,}000 \text{ lbs.}$$

and for the average shear s per lineal foot

$$s = \frac{3{,}080{,}000}{28.5} = 108{,}000 \text{ lbs.}$$

This shear per lineal foot we shall now compare with the shear that could safely be transferred by the lattice construction connecting the ribs of $A\,9\,L$ in a horizontal direction. The ratio which we shall find between both shears must also be the ratio for the moments of inertia I_v and I_h, so that finally I_v can be easily determined.

The lacing angles connecting the ribs of $A\,9\,L$ are riveted to the flanges of these ribs with 12 rivets in the diagonals and with 6 rivets in the vertical angles on each side of the center line, per each lattice panel covering about five lineal feet of the chord piece. If we assume $\frac{2}{3}$ of the rivets in the diagonals and $\frac{1}{2}$ of the rivets in the vertical angles as fully effective in a direction parallel to the chord and call the shear per rivet 4,500 lbs., we get

$$(\tfrac{2}{3} \times 12 + \tfrac{1}{2} \times 6)\ 4{,}500 = 49{,}500 \text{ lbs.}$$

as the safe shear for 5 lineal feet, or very close to 10,000 lbs. per one lineal foot, for which the lacing on the bottom and the top of chord $A\,9\,L$ was good.

The ratio between the shears belonging to the horizontal and the vertical moment of inertia is therefore

$$10{,}000 : 108{,}000 = 0.0926$$

and the moment of inertia which could be made effective by the lacing is

$$I_v = 0.0926 I_h = 0.0926 \times 208{,}000,$$

or $I_v = 19{,}280$.

To this moment of inertia must be added the moments of the ribs, each taken independently, which together amount to about 800. The total effective moment of inertia of $A\,9\,L$ around a vertical axis was therefore close to $I_v = 20{,}000$, in inches, and the corresponding radius of gyration is

$$r = \sqrt{\frac{20000}{781}} = \sqrt{25.5} = 5.1$$

If we substitute now this r in formula (2) and make

$$\left(1 + \frac{\text{minimum stress}}{\text{maximum stress}}\right) = 1.7317$$

as found in strain sheet opposite page 282 of the Engineering News of Sept. 12 we get for the maximum safe working load in $A\,9\,L$ per square inch of section:

$$p = \left(12{,}000 - \frac{50 \times 684}{5.1}\right) 1.7317 = 9{,}170 \text{ lbs.}$$

against $p = 12{,}000 \times 1.7317 = 20{,}780$ lbs., which was adopted for $A\,9\,L$.

It is herewith, I hope, plainly shown that if the column formula prescribed for the structure had been properly applied it would have been clearly demonstrated that the design of the compression number $A\,9\,L$ was faulty and unsafe for the load it was intended to carry.

As formula (2) is modified to comply with Weyrauch's law and gives only a safe working load, it remains of interest to find out what the breaking load of $A\,9\,L$ would have been in accordance with other standard formulas. In the 19th edition, 1905, of the pocketbook "Die Huette," Vol. I., page 374, Euler's formulas are given and their usefulness discussed. The formula most used and best adapted to continuous chords, in which the panel pieces are guided by the panel points of the main trusses, but are not absolutely prevented from swinging around them, is formula II., which reads:

$$(5.)\ P_k = \frac{\pi^2\,E\,I}{l^2}.$$

According to tests made by J. Bauschinger, "P_k is the extreme load under which the small deflections or deviations from the true straight line (which always exist and which under gradually increased loads grow correspondingly) almost at once attain immense proportions," or, in other words, it is equal to the breaking load.

If we introduce in this formula our values and make

$\pi^2 = 10$

$E =$ Modulus of Elasticity $= 30{,}000{,}000$

$I = I_v = 20{,}000$

$l = 684$ ins.

we obtain for the load P_k under which the deflections might suddenly become dangerous

$$P_k = \frac{10 \times 30{,}000{,}000 \times 20{,}000}{684^2} = 12{,}820{,}000 \text{ lbs.}$$

or per square inch of cross-section

$$\frac{12{,}820{,}000}{781} = 16{,}500 \text{ lbs.}$$

which figure is surprisingly close to the unit stress calculated to have taken place in $A\,9\,L$ just before it collapsed. This very close agreement may be a mere coincidence, but it shows at least that my assumption and my reasoning must be generally correct.

Certainly this much can be said, that the longitudinal web shear in compression members of uncommonly large dimensions should be taken care of by something more substantial than mere latticing. Compression members of the sizes used in the Quebec bridge should, by all means, have webs in two directions. This could be attained by forming double T sections with the addition of occasional diaphragms by which the outer flange angles could be stiffened, or by adopting box-shaped members, of which the side plates could be reinforced by additional angles and the like, in the direction of the main stresses.

Respectfully yours,

Alexander von Babo.

1830 Arlington Place, Chicago, Ill., Sept. 17, 1907.

[To assume that the fiber stress in bending K in formula (4), is 16,000 lbs., amounts to saying that all the stress in the member is a bending stress. Such an assumption is of course inadmissible, since only a small part of the extreme fiber stress can or should be due to bend-

ing, so long as the column remains reasonably straight. All the remaining calculations are vitiated by this assumption.

For example, if a column carries a centrally-applied load of 10,000,000 lbs., and is deflected 2 ins., the center moment is 20,000,000 inch-pounds. In the above letter a center moment of 122,000,000 in.-lbs., or twelve times as large as in this deflected state, is used for the calculations. Any condition in which the column would develop so great a moment would evidently be one in which the stage of normal column action has long been passed. To make the calculation of normal strength of the column depend on such a condition seems hardly reasonable.—Ed.]

A Courageous Crawler.

Sir: New Yorkers who use the subway may think there is not so much of courage or disregard of comfort (for a proposed consideration) in the apparently bona-fide offer below, lately received by one of your subscribers. But there is.

In explanation it may be said the pipe referred to is 16 ins. in internal diameter, and nearly five miles in length. It is of wood, and that fact has caused discussion in the neighborhood of the work. Evidently some of this gossip traveled to our correspondent, and in traveling grew, as his letter was the first we heard of any reward. The letter follows:

Kennebunk, Me.

Mr. ———.

Dear Sir: I understand that your company will give $2,000 (two thousand dollars) to any man that will go through a 5-mile pipe for the Purpose of locating a leak. If you have not got anybody yet, I would undertake the contract. Hoping to hear from you, I remain, yours truly, ———.

It is submitted that a few rush-hour subway trips would be a useful preliminary training, and also impress Mr. —— with the horrors of the task he so lightly proposed to undertake.

Very truly, A. H. W.

New York, Sept. 18, 1907.

Contact Area of Wheels and Rails.

Sir: In regard to recent articles on steel rails and rail design, I would say that I have made a great many tests for the area of contact between the wheels and the heads of the rails. The area of contact between driving wheels and rails has been given as an oval about ¾ × 1 in. (or 1 × 1¼ ins.) with worn wheels and tires, and an area of about 1.07 sq. in. These dimensions were usually found on light rails after the wheels were worn, and under static loads. As the stiffness of the rail is increased, so that it does not deflect so much under the wheels, the length of the oval is shortened as a rule, though not in all cases. It has been necessary to widen the head to a slight extent to maintain nearly the same area by increased width. After making a number of tests by jacking up the wheels, and then comparing the width obtained on tissue paper when the wheels are in motion, I am inclined to think that as a rule the area of contact is likely to be greater when the wheels are in motion than when under their static loads.

The results in practice show that the reduced rate of wear on the broad-topped rails must contribute to maintaining a large area of contact, notwithstanding the increased stiffness of the rails. The maximum intensity of the pressure under the wheel contacts would be near the central point, but the metal is so firmly supported on either side that from the cubic elasticity of enclosed metal the elastic limits are not exceeded to as marked an extent as might be anticipated from the linear elasticity of the steel. While we do not know positively that such is the case, a great many tests show that the cubic elasticity or elasticity of volume of sound steel is approximately three times that of its linear elasticity. Therefore metal in the rail head which is confined in the center of the wheel contact area could sustain perhaps 150,000 to 180,000 lbs. without decided flow.

It is of course well known that after the rails are placed in the track and have had some service, the metal is cold rolled to conform to the average wheel tread. Placing a newly turned wheel upon the rails to determine the area of contact does not show as favorable results as the wheels which have had some service and are worn slightly hollow in the tread.

The flow of steel under the wheel treads to the marked extent which is seen upon many rails (forming a lip along the rail head) is due to the fact that from porosity, included slag, or a low grade of metal, the cubic elasticity of the rail head is low. A new feature of investigation in reference to the porosity of the steel (which is indicated by proper etching) shows that where the porosity is high such metal flows quickly under the wheel treads. In other words, with longitudinal "pores" in the head of the rail, the cubic elasticity of such steel is smaller than that of sound metal, and the metal shears readily under the wheel treads.

There is another feature in reference to the flow of metal upon the inside rail of the curves. With the narrow wheel treads of the present day, and especially those which are worn upon narrow-topped rails, the double flange runs upon the top of the rail and causes the metal to shear and flow from the side of the rail. Rails upon curves, instead of being laid to wide gage as is often the custom, should not be increased much in the gage, otherwise the inside rail will be rapidly destroyed by flow of metal from the outside edge.

The area of contact as shown by static loads upon different rails ranges from about 0.25 sq. in. to 0.8 or 0.9 sq. in. as the largest areas measured. The actual wear, however, of the 80-lb. and 100-lb. rails which I made at Scranton with 0.60 to 0.65 carbon (and phosphorus 0.006), after 12 to 15 years of service only show a loss of metal from the top of the head of from 3/32 to 1/8-in. This is a slow rate of wear, and for 100-lb. and 80-lb. rails is a much better result than can be obtained with metal having 0.10% phosphorus. The wear in that class of metal is from three to four times greater than it was with the high carbons and low phosphorus.

I am making a series of examinations of the relative wear, in my studies of the proper metal for rails for our present service.

Yours very truly,

P. H. Dudley, C. E.

80 Pine St., New York, N. Y., Aug. 22, 1907.

An Advocacy of Venturi Meters for Measuring Irrigation Water and of a Premium on Water Economy.

Sir: I was much interested in your editorial, and in Mr. Fortier's article on economy in the use of irrigation water (Sept. 19, 1907). I think, however, that the kernel of the question yet remains to be touched upon: this is, to interest each user of irrigation water *pecuniarily* in saving water. Some time ago I wrote a letter on the subject, which friends of mine thought good enough to put in print in the form in which I enclose a copy.

Very truly yours,

Clemens Herschel.

[The matter sent by Mr. Herschel is dated New York, Dec. 1, 1906, and is entitled "On Measuring Irrigation Water." After stating that under the irrigation projects of the U. S. Reclamation Service each water user will be entitled to a stipulated number of acre-feet of water per annum, measured at his land, or a proportional part thereof in case of a water shortage, Mr. Herschel says that these conditions offer no inducement to save water. He then outlines his experience in the division of water to large consumers while he was connected with the Holyoke (Mass.) Water Power Co. and with the East Jersey Water Co. Broadly speaking, this was that the exact measurement of water distributed in bulk resulted in the curtailment of waste, enabled the water purveyor to increase the number of its consumers, gave it pay for any excess water over the nominal contract rate of supply, and extended the period during which the contract rate of supply was sufficient for consumers.

Coming to the sale of irrigation water by the government, Mr. Herschel urges that water users be induced to save water by providing them with records of actual consumption and by crediting them with under-drafts and charging them with over-drafts. The self-interest of the water user is thus enlisted, and the water saved will make possible an extension or a more intensive cultivation of the area irrigated.

The use of Venturi meters, with straight-reading dials accessible to the water-user and with chart records, also, is suggested. Chart-record registers would cost $350 each, or 25% less in hundred lots. They would furnish a continuous, accurate record of water used, infinitely superior to occasional weir or current-meter gagings, at the same time greatly lessening the necessary engineering supervision of the measuring devices, and substituting for a fixed annual charge, with little regard for the quantity used, a charge dependent on the actual water consumption. It is suggested that by this means the area irrigated by a given quantity of water may be trebled. But, to quote the concluding portion of the letter:

Assume only 20,000 acres, and make the excessive assumption of a meter for every 40-acre plot at a cost, for meters and capitalized cost of operating them, of $250,000.

We should then have 20,000 acres irrigated at a capitalized cost of $37.50 per acre; instead of only 10,000 acres irrigated at a capitalized cost of $50 per acre; the cost of the meters properly forming a portion of the cost of the dam, canals, and the rest of the water-distributing system.

Suppose that it required an expenditure of even $500,000 to induce the water-users to irrigate 20,000 acres with the same quantity of water that is now consumed by 10,000 acres, the cost in both cases being $50 per acre, that is, $1,000,000 to irrigate 20,000 acres or $500,000 to irrigate 10,000 acres, and the money would nevertheless be well spent; for it would yet "cause two blades of grass to grow where one grew before," and thus benefit the state and the nation.

We submit to those of our readers experienced in irrigation matters this proposition to put a Venturi meter on every irrigator, and to give credits for water saved.—Ed.]

Notes and Queries.

K writes:

An interesting thing in connection with the export trade of the Emerson Steam Pump Co., one of our clients, came up while the writer was in Alexandria, Va., a couple of weeks ago. A letter arrived perfectly addressed, originally, to them at Alexandria, Va., U. S. A., and mailed at Calcutta, India. A note, however, made on the envelope by the post-office at Alexandria, Egypt, where it had been sent by mistake, was very forceful in its humor and brevity: "Try America"!

G. W. G., Bellevue, Pa., writes:

I appeal to you for information as to the best method of determining the least radius of a curve around which a locomotive will safely pass. Of course the length of the wheel base and the tread of the drivers are given, the engine is of the type with six drivers and no pony truck. Referring to Trautwine's Pocketbook, 1904 Edition, page 787, he touches on the subject slightly, but speaks of a rigid truck of four wheels. Where will the three axles be located with reference to the radius of the curve?

The builders of the locomotive should have all the information as to clearances of driving axles, etc., necessary to determine the sharpest curve around which the locomotive can pass, and will generally furnish a statement regarding this on request. We know of no satisfactory method of solving the problem other than direct plotting on a large scale plan drawing of the locomotive.

STRENGTH TESTS OF HOOPED CONCRETE COLUMNS.

The University of Illinois tests of concrete columns reinforced by longitudinal rods, reported in our issue of July 11, 1907, pp. 30-33, have been supplemented by an initial series of tests on hooped columns, reported by Prof. A. N. Talbot to the American Society for Testing Materials at Atlantic City, June 22, 1907. The results shown by these tests are, in our view, very important, as they bear on nearly every phase of the employment of reinforced-concrete columns. An abstract of the results is therefore given here.

The tests covered both spiral-hooped and band-hooped columns, together with a parallel set of tests on plain (unreinforced) columns. The concrete was of 1: 2: 4 (stone) mixture, poured quite wet, so that it could be stirred or churned in the forms. The band reinforcement was of three sizes, Nos. 8, 12 and 16 gage, and in all cases 1 in. wide; these bands were welded electrically, and, as sample tests showed failure usually outside of the welds, the bands are to be credited with the full strength of the metal (yield-point about 48,000 lbs. per sq. in.). The spiral reinforcement was of wire, in two sizes, No. 7 gage and ¼-in.; and two grades: ordinary, with yield-point 38,000 and 54,000 lbs. per sq. in., and high-carbon, with yield-point 60,000 and 110,000 lbs. per sq. in. (the higher figure referring to the larger size of wire in both cases). In all cases the hooping was spaced by longitudinal spacing-bars, holding the turns (or bands) generally 2 ins. c. to c.; but in some columns 3 ins. and 4 ins. These spacing-bars, however, were hardly capable of taking any longitudinal compression, and were in no case included in the calculations as reinforcement. The quotient: Volume of hooping steel divided by volume of concrete, was taken as the reinforcement ratio (and multiplied by 100 to give percentage). The diameter of the bands and spirals was made such that the concrete barely covered them; this eliminated any contributory effect of concrete outside of the hooping.

The test columns were round, generally 12 ins. in diameter by 10 ft. long; a few were 9 ins. in diameter. They were molded in galvanized-iron forms, on cast-iron base plates. The forms were removed after 10 days, and the columns were tested when about 60 days old. A few days be-

fore testing, an upper cast-iron plate was fixed to each column by means of plaster-of-paris.

The set of unreinforced columns provided to give a basis of reference showed test results substantially the same as the results heretofore obtained. Those which were of 1:2:4 mixture averaged about 1,620 lbs. per sq. in., ranging 30% either side of this figure. Fig. 1 shows a typical stress-deformation diagram. As in previous tests, the curve of longitudinal stress plotted on longitudinal compression approximates a parabola. Measurements of the transverse expansion, or bulging, of the column were also made. These are plotted in the curve at the left in Fig. 1. Prof. Talbot says concerning them:

> The measurement of this expansion is extremely difficult, and this work is of an experimental nature and the results are not to be considered final. Further study of the data will be required to interpret them fully. It seems, however, that the value of Poisson's ratio found from these tests is generally less than that given in text-books. This is particularly true for the smaller loads. The ratio increases rather suddenly somewhat before the maximum strength of the column is reached. From a hasty study of the results it would appear that Poisson's ratio averages 0.10 or somewhat more, for the lower loads, and that near the maximum load this ratio rapidly increases to 0.25 or 0.30.

Another matter incidental to the tests on plain columns was the determination of the influence of richness of concrete. The results of failure tests of columns made with various mixtures yield the instructive plot reproduced as Fig. 2, which suggests practically a straight-line relation between amount of cement and compressive strength. The remarkable closeness of the plotted points to the line contrasts radically with the character of other plots of cement or concrete tests. This cannot be due to the inclusion of an exceptionally large number of individual tests in the four plotted averages, as the paper from which we draw the present abstract says that the "results given are the average of from 2 to 5 test columns."

Turning now to the tests of hooped columns, we reproduce as Fig. 3 a load-deformation curve presented as typical, and in Fig. 4 a photograph view of one of these columns taken after the maximum load had been passed, and when most of the outside scale of concrete had split and fallen off. The appearance of this column, with its lateral buckling similar to that of a fixed-end steel column, clearly suggests the high degree of compressibility which forms the characteristic feature of the hooped column. However, more important, though less striking, features are contained in the diagram, Fig. 3. Quoting from the paper:

> The general phenomena of the tests of hooped columns are as follows. The early part of the test is much the same as for plain concrete columns. At a load equal to that which would cause failure in a plain concrete column, or a little above, the concrete over the spacing begins to scale, and this is soon followed with a scaling and shelling off of the surface of the column over the hoops everywhere. With added increments of load the amount of shortening increases rapidly and the column expands or bulges laterally correspondingly. The lateral deflection of the column from a straight line begins to be apparent just before the maximum load carried by the column is reached, and it rapidly increases after the maximum load is passed, forming a curve having the characteristics of the figure shown in text-books for columns with fixed ends. The columns finally bent out of line, in some cases as much as 4 or 5 ins., the load finally carried being only a small part of the maximum.
>
> The load carried when the column had shortened an amount equal to that which plain columns exhibit at failure corresponds to the maximum load of plain concrete columns. If [in Fig. 3—Ed.] the line beyond the elbow (approximately straight) is produced to the vertical axis or axis of ordinates, the load there indicated agrees closely with the maximum strength of plain concrete. The diagram indicates that up to the load where a plain concrete column would fail the shortening and the lateral deformation correspond closely with those obtained for plain concrete columns, and but a small stress is developed in the hoops. The line at the left of the figure gives the lateral unit-deformation. Beyond the critical point referred to, the longitudinal determination or shortening increases rapidly with an increase of load and the lateral deformation increases correspondingly. The total amount of shortening is considerable, in this particular column being 9 times as much at the maximum load as may be expected in a plain concrete column at its maximum load. The maximum strength of the columns occurs when the bands have been stretched to their yield-point. Beyond this point the deformations increase rapidly, but the load falls off.

The effect of the hooping is very simply summarized by Prof. Talbot in the words:

> The additional strength over that of plain concrete columns is a function of the amount and strength of the reinforcing,

in connection with which it is important to remember that up to the load at which an unreinforced column would fail, the hooping apparently takes no part in the action, or at the best a negligible part.

Fig. 5 is a comparative plot showing the relation between strength and amount of reinforcement. Prof. Talbot has drawn two straight lines, for low steel and high steel, respectively, to represent the results mathematically; the individual points show material departures from these curves, however.

Comparing the tests of hooped columns with the tests of columns using longitudinal reinforcement, Prof. Talbot remarks that apparently the strength added by a given amount of steel, when the steel is used in the form of hooping, is from two to four times as great as when the steel is used in the form of longitudinal reinforcement.

The *combination* of hooping with longitudinal

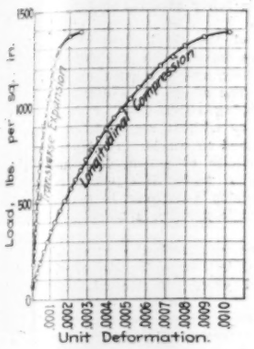


Fig. 1. Stress Deformation Curve of Unreinforced Concrete Column.

(Diameter of column, 12 ins.; Length, 10 ft. Concrete, 1:2:4, 60 days old. Maximum load, 156,000 lbs.)

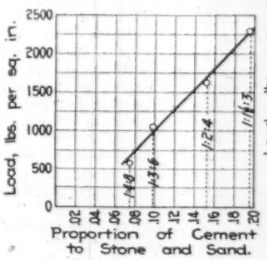


Fig. 2. Variation of Compressive Strength with Richness of Mixture in Unreinforced Concrete Columns.

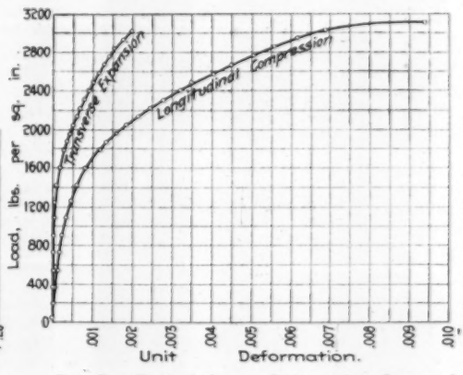


Fig. 3. Typical Stress Deformation Curve of Hooped Concrete Column.

(Diameter of Column, 12 ins.; Length, 10 ft. Concrete, 1:2:4, 60 days old. Reinforcement, 60 hoops of No. 12 steel 1-in. wide, = about 2.12%. Maximum load, 351,000 lbs.)

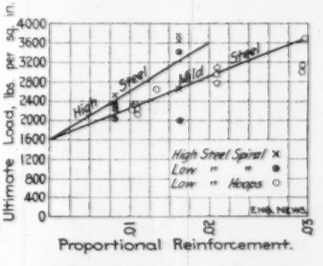


Fig. 5. Variation of Compressive Strength with Percentage of Hooping Reinforcement in Hooped Concrete Columns.

Approximate formulas:

Mild steel,

$$p = 1{,}600 + 65{,}000 \times \frac{\text{percentage}}{100}.$$

High steel,

$$p = 1{,}600 + 100{,}000 \times \frac{\text{percentage}}{100}.$$

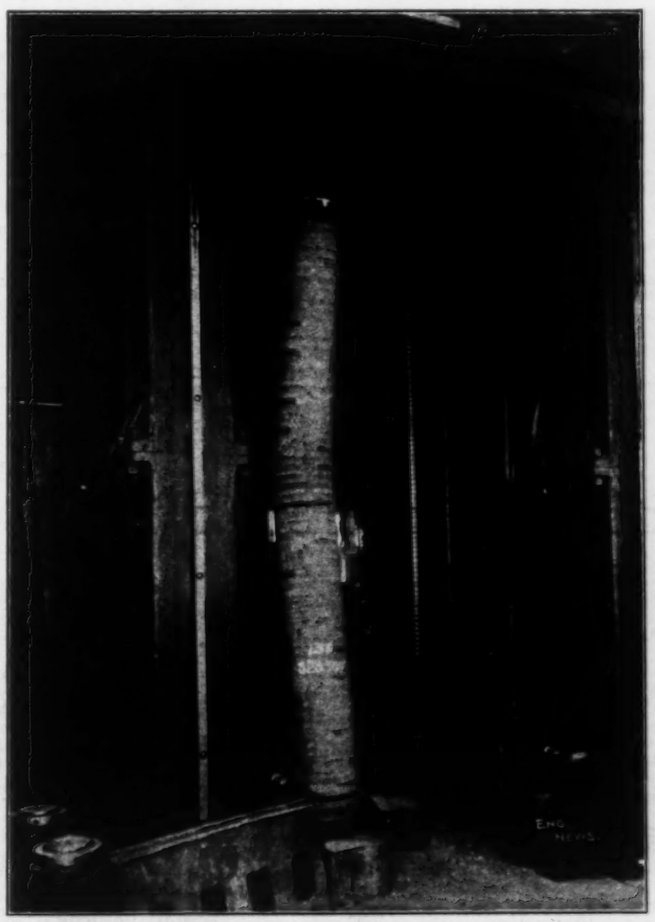


FIG. 4. VIEW OF HOOPED CONCRETE COLUMN UNDER TEST, SHORTLY BEFORE FAILURE.

(Maximum load already passed. Outer shell of concrete has fallen off.)

reinforcement, in the same column, is not desirable, since the longitudinal steel would reach its yield-point long before the hooping, and, in consequence, either the full strength of the hooping could not be utilized or else the longitudinal reinforcement would be much overstressed and distorted by the time the hooping reached its limit of elasticity.

This question is, however, characterized as "uncertain" by Prof. Talbot himself. It is bound up with the question of the physical state of the concrete during the second stage of the hooping, i. e., when the hooping is in action. Two steps were taken toward a study of the latter question. First, on the supposition that the hoops are stressed by something like internal fluid pressure, calculation from the results of the tests above recorded indicate a lateral pressure amounting to about 0.35 of the excess of longitudinal pressure over the strength of the concrete itself. Second, to actually examine the condition of the concrete within a hooped column during this stage of loading, the following experiment was performed:

A spiral hooped column

was loaded to 2,000 lbs. per sq. in. and showed a unit shortening of 0.0036. The load was released and a set of 0.0025 was found. A load of 2,000 lbs. per sq. in. was again applied, the unit-shortening becoming 0.0041, and then released. The spiral was then stripped from the column without taking it from the machine, and the naked column failed at 1,080 lbs. per sq. in. As the indications of the stress-deformation diagram are that the plain concrete would have held 1,200 lbs. per sq. in. on first application of the load, it may be considered that the concrete sustained 90% of its original strength. The hooped column itself might have held 2,200 or 2,400 lbs. per sq. in.

Returning now to the principal purposes of the tests of hooped columns, we quote the following series of conclusions drawn by Prof. Talbot. It must be remarked, however, that the author expressly says that much yet remains to be learned about hooped concrete, and that the conclusions are given only tentatively.

(1.) Poisson's ratio for concrete in compression is a variable quantity, increasing considerably just before the ultimate failure of the concrete. A value of 0.1, or somewhat more, may be tentatively given for the lower loads, and 0.25 to 0.3 near the crushing load. This concrete set in air.

(2.) In hooped columns the hoops do not come into action to any great extent before a load equivalent to the ultimate strength of plain concrete, or a little below, is reached. The longitudinal deformation and lateral deformation of the concrete is not modified by the hooping to any great extent before this load is reached.

(3.) Before this point both the longitudinal deformation and the lateral deformation are approximately proportional to the added load. The ratio of lateral to longitudinal deformation is more nearly constant, and in these experiments the ultimate strength of the column is controlled by the elastic limit of the hooping.

(4.) The total amount of shortening before failure occurred was very great, averaging something like 8 or 10 times that for plain concrete columns, and 60 times that for the ordinary working stress in plain concrete. The longitudinal shortening is, say, 8 times that of mild steel at its elastic limit. Cracking and peeling of the concrete appear at loads corresponding to the ordinary ultimate strength of concrete.

(5.) The excessive amount of shortening before failure affects the problem of combining hooping and longitudinal reinforcement very unfavorably, if the stresses are to be kept within the elastic limit of the latter.

(6.) The lateral deflection of hooped columns is large and may affect the ultimate strength which is available for the column. For continued application of stress beyond the maximum load, the column deflects enormously. Scaling of surface of concrete, and lateral deflections, are warning signs given well before danger of failure exists.

(7.) The concrete itself retains a considerable element of its strength even after it has shortened in a hooped state four or five times as much as would produce failure in unhooped concrete columns.

(8.) Columns of richer and leaner concrete exhibited phenomena of similar characteristics, the hoop stress becoming effective at the ultimate strength of unhooped concrete.

(9.) Hoops of high elastic limit steel give greater strength than these of mild steel, but the increased amount of shortening necessary to develop the full strength of the column is an undesirable feature.

(10.) One experiment indicates that hooped columns will resist eccentric stresses in somewhat the same way as will other material.

(11.) Light hooping offers security against sudden failures and unevenness of concretes, and will enable higher working stresses to be used. In combination with rich concretes and longitudinal reinforcement, using low stresses in the hoops (i. e., basing the strength upon an assumed ultimate strength but little beyond the average ultimate strength of plain concrete) a satisfactory column may be made. It is suggested that a column of this character may be designed in such a way that the longitudinal reinforcement may carry the load during construction and still not be overstressed later for the additional stress which goes with the concrete, provided the basal point for the column strength is somewhat below the ultimate strength for plain concrete.

(12.) Heavy hooping gives added strength, but in utilizing the full strength of such columns the column shortens unduly, deflects laterally, and will strain longitudinal reinforcement many times beyond the deformation which exists at the elastic limit of the metal. It may be applicable where a large limit of safety is desired, or where large variations in shortening are unobjectionable, as where the structure is articulated. So far as ultimate strength is concerned, hooping adds two to four times as much strength to the column as does an equal amount of longitudinal reinforcement.

In these conclusions no direct mention is made of "set" or permanent compression. It is known that a hooped column not only undergoes a very great shortening during test, but also exhibits very large set in that stage of loading where the hoops are in action. This phenomenon will undoubtedly have much influence in defining the limits of working stress in hooped columns. For example, if, in order to bring the hoops into action at all, the column must suffer large permanent distortion, which cripples it for continued useful service under most conditions; then the hooping can be looked on only as a reserve of strength against sudden collapse. In this case, that part of the margin of safety which covers ordinary variations must be applied to the strength of the unreinforced concrete, leaving to the hooping only the function of a reserve of safety.

The tests above recorded doubtless include complete measurements of set at various loads, although the diagram as well as the text make no reference to such measurements. The future publication of the set observations may therefore be looked to with the certainty of their important bearing on the subject of hooped columns.

THE PERMEABILITY OF CONCRETE AND METHODS OF WATERPROOFING.*

By RICHARD H. GAINES.†

The following paper is the result of an investigation now being carried on by the New York Board of Water Supply in their effort to perfect the materials and methods of construction in the immense new aqueduct for the City of Greater New York.—[Ed.]

INTRODUCTION.—The preliminary work relating to the subject embraced a comprehensive study of cement and concrete as a whole, including the materials, composition, testing, proportioning and observation of practical experiments in the physical laboratory. Attention was then directed to the permeability of concrete with a careful examination of the best known methods of waterproofing in present use. The methods hitherto devised may be roughly classified as follows:

(1) Processes involving the use of asphalt or other bituminous compounds, directly applied to the surface, or through any medium, the fundamental idea being to prevent the water from coming in contact with the concrete.

(2) Washes applied to the surface of the hardened concrete; the purpose being to fill the surface pores with some insoluble compound by precipitation.

(3) Substances added to the cement, or to the mortar at the time of mixing, hence forming an integral part of the concrete. The principle here is to fill the pores of the mass with some finely divided insoluble material.

Owing to the fact that asphalt and the compounds belonging to this class are impervious to, and insoluble in either salt or fresh water, methods in which these materials constitute the base have been largely used for waterproofing for many years. A difficulty about these processes is that if put on directly, or through the medium of some bituminous paint as a binder, they can only be applied to dry surfaces, and thus are not adapted to certain kinds of work. On the other hand, good authorities object to the use of felt of any kind in connection with underground waterproofing, especially about joints, owing to the practical impossibility of getting the felt tight. The greatest objection to the asphalt processes, however, is that their durability for any considerable period is still an open question. It is a few years at best the most of these bituminous substances oxidize, become brittle and porous, break down, and thus after a time cease to be effective for waterproofing.

Among the surface methods, the old Sylvester or soap and alum process, is fairly effective, but is very expensive for large work.

The examination of the third class of processes, in which materials are mixed with the cement, included absorptive, permeability, and strength tests, and analyses of the compounds. The results showed that some of the so-called "waterproofing compounds" were worse than useless. Put together without regard to chemical principles, they were found to consist in large part of inert substances, the particles of which, in cement mortar, acted as impermeable septa, interfering with normal setting and thus causing a decrease in tensile and compressive strength, while exerting a negative influence on permeability.

Several of the processes examined were apparently based on sound principles, but were far too expensive to be seriously considered except in special cases and for limited use.

The statement is made by one well-known authority on cement and concrete that additions of from 5 to 10% of hydrate of lime, replacing such proportions of cement at the time of mixing the mortar, will produce practical impermeability. This, however, is not in accord with our experience. Our experiments indicate that, while slaked lime slightly decreases the permeability, this advantage is more than offset by the decrease in the strength of the mortar. There is no chemical reason why other results should be expected.

Without going further into the merits or demerits of methods of waterproofing that have found more or less favor among engineers, it is sufficient to say that none of the processes examined gave results that would justify recommendation for extensive use.

OLD THEORIES AND NEW PRINCIPLES.—In the study of the waterproofing problem it was first necessary to determine, if possible, what actually happened in the wet mass. The accepted chemical theory is that the finely ground calcined mixture of carbonate of lime and clay, or argillaceous limestone of suitable composition, when mixed with water, solidifies and hardens, owing to the formation of hydrated aluminum calcium silicates, and basic calcium aluminates. During the chemical interchanges, calcium hydroxide is liberated, and if the amount of water present is limited, it soon becomes saturated with respect to this base, which crystallizes out, to be again replaced with more hydroxide, until the cement is hardened, silicates being left of a lower degree of basicity.

From a purely chemical viewpoint, the whole question of setting is still *in nubibus*, and the precise changes that take place will continue to offer a fruitful field for theory.

In the discussion of the decomposition of cement we are never dealing with homogeneous materials, but a complex system — a group of bodies which undergo a plurality of chemical and physical changes, occurring either at the same time, or following each other, step by step. Such reactions can, without mutual interference, take place side by side, just as sound, light and electric waves, or currents of heat, electricity and diffusion can pass through a medium simultaneously. The silica + lime + alumina + X in cement become, on the addition of water, an unstable system, which at once begins to rearrange itself. Several reversible reactions take place, involving dissociation, followed slowly by reassociation. During this process of change and readjustment to a state of equilibrium, the unstable substances that are formed as intermediate products are not known separately, and their exact nature can only be conjectured.

Perhaps the meaning of the term "equilibrium" here should be made clear. Every system of bodies strives to approach a state in which the amount of energy which can be transformed into work (available energy) is as small as possible. When a chemical system reaches this state it is said to be in equilibrium. Every such state is really not to be regarded as a static but as a dynamic system, since the transformation of materials does not cease in a state of equilibrium, but the tendencies in both directions compensate one another so that, as a result, no further alteration in the given chemical system can be detected.

After a careful survey of the unsatisfactory laboratory results obtained with the previously known processes of waterproofing concrete, the problem was attacked from a new standpoint. A pioneer attempt was undertaken to apply some of the theories and methods of physical chemistry to its solution.

APPLICATION OF THE ELECTROLYTIC THEORY.—The amalgamation of the sciences of physics and chemistry during the last two decades has afforded the means of solving many complex problems. The ionic hypothesis,

*Portion of the Discussion on Paper presented, April 17, 1907, to the American Society of Civil Engineers, by Messrs. W. B. Fuller and Sanford E. Thompson, on "The Laws of Proportioning Concrete." Reprinted from the Transactions of the Society, Vol. 59, p. 67.

†Chemist, New York Board of Water Supply.

conceived by Faraday, and brilliantly developed by the great Swedish physicist Arrhenius, connecting the facts of electrolytic changes with those of chemical interaction, marks a new era in our molecular conceptions, since it has revealed an entirely new species of molecule, viz., the electrically charged ion. With the help of this ingenious theory, the science of chemistry has undergone a renaissance, and many long observed, but little understood, phenomena have found explanation.

It has long been known (first observed by the Rogers brothers in the last century) that when certain finely powdered minerals are treated with water, reactions take place which result in partial decomposition. Concerning this fact, investigation with the microscope has shown that water produces a colloid or glue-like surface condition of the particles leading to plasticity in the wet mass, and increasing the binding power or induration. The last change is due to the surface of the particles becoming clogged with the product of hydrolytic decomposition. This general process is precisely similar to what follows mixing water with cement; but the reactions go farther in the latter case.

If we interpret the complex series of changes that take place in cement by the electrolytic dissociation theory and considerations developed in this study, the conclusion is arrived at that the following consecutive processes take place in the wet mass:

(1) Hydrolytic dissociation, i. e., dissociation under the action of the ions of water.

(2) Formation of colloidal films about the particles that act as osmotic membranes, permeable to the ions of electrolytes.

(3) Absorption of basic ions by the hydrogels, resulting in coagulation.

(4) Recombination of dissociated molecules tending to chemical equilibrium.

FUNCTIONS OF ELECTROLYTES.—After some experiment and much study of the problem of waterproofing concrete, it was believed that if the decomposition reactions could be made to go further, i. e., with more reactivity in the mass, the hardened mortar would not only be rendered impervious, but, owing to the greater density, would have increased strength. The experiments of A. L. Cushman on the decomposition of feldspars gave a valuable clue here. He found that treatment of powdered feldspars with dilute solutions of various salts (electrolytes) leads to an increased amount of decomposition products passing into solution. He obtained the same quantity of soluble decomposition products from a finely ground feldspar by treatment with a dilute solution of ammonium chloride as could be extracted by electrolysis without subsequent regrinding. Cushman also found that the slowing up of hydrolysis upon finely ground feldspar is due to the clogging effect of insoluble aluminum silicate which is precipitated in the colloidal form on the surface of the particles. He concluded that the effects produced by dilute solutions of salts was principally to set free the soluble products of hydrolysis absorbed by the colloidal decomposition products, and thus expose fresh surfaces to the solvent.

The germ of truth thus laid bare suggested that the decomposition reactions in cement would be augmented in the same manner, namely, by replacing the mixing water with a very dilute solution of a suitable electrolyte—some substance, the free charged ions of which would bring about a rearrangement of the molecules of the bodies with which they came in contact, causing a complete change in the equilibrium conditions of the system. The desired chemical and physical changes might be induced in part by catalysis or accounted for as either electrical or chemical force. Such actions, however, are usually regarded as merely manifestations of the affinities of one kind of matter for another.

We do not yet know the exact nature of chemical affinity. Berzelius long ago asserted the existence of a close connection between electrical and chemical forces. We know that chemical reactions produce electricity, and conversely that electricity often causes chemical action. The starting of a chemical change may come about through the agency of some very slight external influence in the form of electrical action. Clausius found that the smallest conceivable electric "force" is sufficient to effect the decomposition of an electrolyte. This involves the loosening of the chemical energy, since a part of the molecules must first split into atoms, or the union of atoms in the molecule must first be severed before the chemical reaction becomes possible. The reaction, once started, proceeds of itself.

These well-known facts are repeated to emphasize the intimate relation between chemical and electrical action, and furthermore, the close relationship of matter and electricity. Indeed, the present day idea among many physicists is that all homogeneous substances are synthesized from atoms which are collections of positive and negative charges held together mainly by their electrical attractions. Electricity is supposed to have an atomistic structure, with positive and negative elementary particles which are about thirty thousand times smaller than the atoms of ordinary matter. Just as free ions have an existence, so do free electrons have a momentary existence.

In our views of the phenomena of matter and electricity, we are apparently in a transition state between the atomic hypothesis and the electro-corpuscular theory, and these new lines of thought should not be lost sight of in any physical investigation.

In every chemical system there is a tendency to undergo change of some definite character. The conditions under which the system exists may be such that the affinities are in a state of stable equilibrium among themselves. Every system may be regarded as having a point of least resistance at which an alteration will most easily take place. If a suitable material were introduced into a chemical system it might react with certain constituents of the system to such an extent that what was before merely a tendency to undergo change would become an actual change beginning at the point which, before the introduction of the material, was the weakest point of the system.

Applying this theory to the decomposition of cement, it seemed probable that the electrolyte would form with some constituent of the system compounds too unstable to exist under the conditions, which compounds would immediately break up, leaving the ions of the electrolyte free to react with fresh portions of the system, a process repeating itself to a state of equilibrium. The presence of the electrolyte would therefore cause a very different set of conditions from that existing in its absence—conditions that might easily bring the desired properties of impermeability along with increased strength to concrete. To summarize, it was believed that the introduction of the electrolyte would cause this result:

(1) By changing the equilibrium conditions of the system.

(2) By increasing the quantities of reacting substances in the mass. (The fundamental idea of the "Law of Mass Action.")

(3) By setting free the basic ions absorbed by the colloidal decomposition products, thus exposing increased surface to the action of the solvent.

(4) By catalysis, or electrical action set up in the mass by the excess of free ions, thus inducing chemical action.

Experiments based on these conclusions, with tabulated results, will be found accompanying this paper.

COLLOIDS.—While pursuing the study of materials that might be adapted to rendering cement mortar waterproof, attention was early attracted to the properties of certain colloidal substances. The important role played by the colloidal decomposition products of cement had already been observed and pondered over. There were reasons for believing that the process of setting was largely due to the coagulation of inorganic colloids, and the slowness of the action was in consequence of the indolence with which changes are prone to take place in these bodies. It seemed hardly possible that simple crystallization of calcic hydroxide or other compounds could give rise to the great strengths. These considerations led to a somewhat extended study of the properties of colloids, having in view a series of permeability and strength tests with additions of finely ground colloidal clay in cement mortars. (The results of these experiments will be found in the accompanying table.)

Substances existing in solution fall into two groups, the crystalloids and colloids, the physical and chemical behavior of which are characteristically different. Crystalloidal substances easily assume the crystalline condition, forming porous masses. The colloids are incapable of crystallizing, thus proving that the two classes of bodies are wholly different in molecular structure.

Many metallic oxides, and other inorganic compounds, such as are found in plastic clay, for instance, have the power of entering into that peculiar hydrated, non-crystalline condition which Graham called "colloid." Later Van Bemelen called the inorganic colloids "hydrogels." These curious substances consist of heterogeneous molecular complexes which possess a submicroscopical, web-like formation, the distinguishing characteristic of which is the peculiar relation to water they exhibit. Dried at moderately high temperatures up to a certain critical point, they will lose nearly all their water, only to take it back again eagerly when allowed to cool in free air or in moist atmospheres. This dehydration and rehydration can be repeated indefinitely unless the temperature of drying is carried too high, when the property is gradually lost and finally destroyed. A change in state impresses itself upon colloids and determines the conditions in which these bodies will be ultimately found. In other words, a colloid seems to remember more or less perfectly a change it has suffered, just as does living matter.

Many colloidal solutions are able to coagulate either on the addition of foreign substances or spontaneously. What the nature of the process is, and whether it is comparable to crystallization, cannot be stated. A coagulated solution presents several peculiarities—an intermediate thing between a solid and a liquid, its enormous internal friction relates it to solids. The simplest conception of a hydrogel is that a solid substance is separated which has a "network" structure, the interstices of which are filled with water, which is held fast by capillary forces.

The properties of substances which assume the colloidal state are in part due to the approach of each particle to its molecular state, with a resulting proportional increase of active surface. If we imagine certain inorganic substances divided under water into smaller and smaller particles until this dust is able by virtue of its minuteness to remain suspended in the liquid an indefinite length of time, then we have a colloidal solution. Such a solution represents a suspension of fine particles, electrically charged from giving off ions, just as are electrodes. If now, the suspended particles become steadily smaller and smaller while at the same time their electric charge grows, they approximate more and more the behavior of ions, until finally the colloid passes over into the crystalloid, which dissociates in aqueous solution into its strongly charged ions. It can easily be seen how through changes in only a few conditions an enormous variety in the character of the colloidal reactions is brought about, a behavior apparent in numerous transitions between colloids and crystalloids.

Other physical criteria of colloidal solution are: sensitiveness to an electrolyte, and movement in an electric field. Colloids when placed in a field of force migrate, some to the cathode and some to the anode. In this respect they behave as electrically charged particles. Thus ferric hydroxide is a positive colloid, and moves to the cathode, and silicic acid a negative colloid and moves to the anode. Electrolytes cause the precipitation of colloidally dissolved substances, and the degree in which this property is manifested depends on the electric charge of the ion rather than on the chemical nature of it.

Not only can electrolytes cause the precipitation of colloids, but one colloid can also cause the precipitation of another. In this case there is a certain optimum concentration of the colloid. When a colloidal precipitate is formed in the presence of an electrolyte, a portion of the cations (basic ions) of the electrolyte is absorbed by, or adsorbed on the colloidal matter, in the form of the hydroxide, while the anions (acid ions) remain in solution. Cushman has shown that in certain cases the base will be absorbed from solutions of electrolytes by colloid substances, the acid ion remaining free in solution. In other cases the acid will be taken and the base left, and again the whole salt will be occluded. This absorption power is in no sense due to chemical combination, but seems to depend upon a submicroscopic physical property of the colloid. Furthermore, there is evidence which points to the cause of the phenomenon as due to electro-static conditions maintained between

TABLE SHOWING EFFECT OF ELECTROLYTES AND COLLOIDS UPON STRENGTH AND PERMEABILITY OF CONCRETE.

Mortar				Strength in pounds per square inch. Compressive.				Tensile				Permeability. Grams of water passing in last 10 minutes of test.				
Atlas cement, parts.	Cow Bay sand, parts.	Fine clay.	Alum solution.	7 days.	28 days.	90 days.	Specimens in average.	7 days.	28 days.	90 days.	Specimens in average.	40 lbs. pressure.	Age in days.	80 lbs. pressure.	Age in days.	Specimen No.
1—REPLACING THE MIXING WATER WITH A DILUTE SOLUTION OF AN ELECTROLYTE.																
1	3	....	....	1180	1635	1635	3	155	205	272	6	527	28	524	28	1
												90	....	33	....	2
1	3	....	2½%	1105	2010	2230	3	206	229	289	6	62	28	10	30	1
												0	....	5	....	2
1	3	....	5 %	1025	1615	1815	3	181	227	298	6	Trace	29	2	30	1
												Trace	....	2	31	2
2—THE ADDITION TO THE MASS OF COLLOID IN THE FORM OF COLLOIDAL CLAY.																
1	3	....	....	1180	1635	1635	3	155	205	272	6	527	28	524	28	1
												90	....	33	....	2
1	3 (95% + 5%)		....	1590	2390	3345	3	198	258	328	6	....	....	....	....	....
1	3 (90% + 10%)		....	1895	2350	3810	3	270	335	382	6	0	30	0	32	1
												0	....	0	....	2
3—REPLACING WATER WITH ELECTROLYTE AND ADDITION OF COLLOIDS TO THE MASS.																
1	3	....	....	1180	1635	1635	3	155	205	272	6	527	28	524	28	1
												90	....	33	....	2
1	3 (95% + 5%)		2½%	1605	2445	3545	3	218	269	358	6	0	28	Trace	30	1
												0	....	....	....	2
1	3 (90% + 10%)		5%	1855	2945	3130	3	262	342	414	6	0	28	Trace	30	1
												0	....	....	....	2

the ions of the electrolytes and the surface charges of the particles.

The colloids are an excellent means, under certain circumstances, of keeping slightly soluble salts in solution, in that they themselves unite with salt ions and form molecular aggregates without necessarily being precipitated. In the setting of cement this is a most useful function of colloids. By this means the precipitation of the scarcely soluble salts is retarded or suppressed, so that the action can take place in the time desired, and with a gradual evolution of heat. The function is similar to that performed by a negative catalyser and is of vital importance, remembering that the amount of available energy in the form of heat produced in a chemical reaction is always the same, whether the reaction takes place slowly or not.

The hardening of cement is probably not so much a chemical as a physical process, and may not be due to the formation of chemical compounds at all, but to a specific interaction of colloids and electrolytes which protects the calcium compounds from quick hydration, with consequent sudden increase in volume, due to rise in temperature. The deduction follows that without a properly balanced mixture of colloids and electrolytes in the mass, a theoretically perfect cement in strength and impermeability would be impossible.

In all these considerations a proper amount of mixing water for optimum setting conditions is, of course, assumed. It must be remembered that a substance in solution behaves as a gas, occupying the same volume as the solution, and the laws which substances in solution obey are analogous to those which are obeyed by gases—(Van't Hoff's law). Hence, an excess of water weakens cement because the spaces between the nidi, or the particles of the active mass, become too great for uniform coherency in the hardened product.

These studies suggested a series of permeability and strength tests on cement mortars, with additions of different quantities of a suitable electrolyte and some form of colloidal matter, or both. The electrolyte chosen was ordinary alum, dilute solutions of which were used to replace the mixing water (2½% and 5% solutions). The form of colloid selected was that contained in specimens of finely ground clay. Many clays contain a large amount of substances that assume the colloidal state on the addition of water, which sets free gelatinous decomposition products by hydrolysis.

For the results obtained on permeability recorded in the table on page 545, 2-in. cubes were used; for tensile strength, ordinary standard briquettes, and for compressive tests, 2-in. cubes.

SUMMARY.—The results from this series of experiments indicate that watertight concrete may be obtained in three ways:

(1) By replacing the mixing water with a dilute solution of a suitable electrolyte. The concentration of the solution need not exceed 2% in strength. A 1% solution is believed to be sufficient, since greater concentrations are not further ionized.

(2) By replacing from 5 to 10% of the cement with an equal quantity of dried and finely ground colloidal clay, intimately mixed with the cement.

(3) By the use of both the solution of the electrolyte and the clay.

The tables show that these substitutions not only give a watertight product, but, for the periods of seven to ninety days, give greatly increased strengths.

THE INTERSTATE COMMERCE COMMISSION'S STATISTICS OF RAILWAYS IN THE UNITED STATES FOR THE YEAR ENDING JUNE 30, 1906.

In our issue of Sept. 5, we printed statistics of United States railways from the advance figures furnished by the publishers of "Poor's Manual." Below we present some of the most interesting figures in the statistical report of the Interstate Commerce Commission, soon to be issued:

MILEAGE.—On June 30, 1906, the report shows that the total single-track railway mileage in the United States was 224,363 miles, or 6,262 miles more than at the end of the previous year. The aggregate length of railway mileage, including tracks of all kinds, was 317,083 miles. This mileage was thus classified: Single track, 222,340 miles, as just mentioned; second track, 17,936 miles; third track, 1,766 miles; fourth track, 1,279 miles, and yard track and sidings, 73,760 miles. These figures indicate that there was an increase of 10,286 miles in the aggregate length of all tracks, of which 3,819 miles represented the extension of yard track and sidings.

EQUIPMENT.—On June 30, 1906, there were in the service of the carriers 51,672 locomotives, the increase being 3,315. These locomotives, excepting 1,090, were classified as: Passenger, 12,249; freight, 29,848, and switching, 8,485.

The total number of cars of all classes was 1,958,912, or 116,041 more than for the year 1905. This rolling stock was thus assigned: Passenger service, 42,262 cars; freight service, 1,837,914 cars, and company's service, 78,736 cars. These figures do not include cars owned by private commercial firms or corporations.

The average number of locomotives per 1,000 miles of line was 232 and the average number of cars per 1,000 miles of line was 8,814. The number of passenger-miles per passenger locomotive was 2,055,309, showing an increase of 6,751 miles as compared with the previous year. The number of ton-miles per freight locomotive was 7,232,563, showing an increase of 541,863 miles.

The number of locomotives and cars in the service of the railways aggregated 2,010,584, of which 1,827,789 were fitted with train brakes, or an increase for the year of 186,394, and 1,989,796 were fitted with automatic couplers, or an increase of 118,206. Of 1,837,914 cars in the freight service on June 30, 1906, the number fitted with train brakes was 1,689,141, and with automatic couplers 1,820,854.

EMPLOYEES.—The reported number of persons on the payrolls of the railways in the United States on June 30, 1906, was 1,521,355, which is equivalent to an average of 684 employees per 100 miles of line. These figures show an increase in the number of employees as compared with the year 1905, of 139,159, or 47 per 100 miles of line. The total number of railway employees, disregarding a small number not assigned, were apportioned among the four general divisions of railway employment as follows: For general administration, 57,054; for maintenance of way and structures, 495,879; for maintenance of equipment, 315,952, and for conducting transportation, 649,820.

The total amount of wages and salaries reported as paid to employees during the year ending June 30, 1906, was $900,801,653; but this amount is deficient by more than $27,000,000 because of the loss of railway records in the San Francisco calamity.

CAPITALIZATION OF RAILWAY PROPERTY.—On June 30, 1906, the par value of the amount of railway capital outstanding was $14,570,421,478, which is equivalent to a capitalization of $67,936 per mile for the railways in the United States. Of this capital there existed as stock $6,803,760,093, of which $5,403,001,962 was common and $1,400,758,131 preferred; the remaining part, $7,766,661,385, represented funded debt, consisting of mortgage bonds, $6,266,770,962; miscellaneous obligations, $973,647,924; income bonds, $301,523,400; and equipment trust obligations, $224,719,009.

Of the total capital stock outstanding, $2,276,801,333, or 33.46%, paid no dividends. The amount of dividends declared during the year was $272,795,974, being equivalent to 6.03% on dividend-paying stock. For the year ending June 30, 1905, the amount of dividends declared was $237,964,482.

PUBLIC SERVICE OF RAILWAYS.—The report indicates that the number of passengers carried by the railways in the year ending June 30, 1906, was 799,507,838, this item being 60,673,171 more than for the year ending June 30, 1905. The passenger-mileage was 25,175,480,383, the increase being 1,375,330,947 passenger-miles.

The number of tons of freight shown as carried (including freight received from connections) was 1,631,374,219, which exceeds the tonnage of the year 1905 by 203,642,314 tons. The ton-mileage, or the number of tons carried 1 mile, was 215,877,551,241, the increase being 29,414,441,731 ton-miles. The number of tons carried 1 mile per mile of line was 982,401, indicating an increase in the density of freight traffic of 121,005 ton-miles per mile of line.

The average revenue per passenger per mile for the year ending June 30, 1906, was 2.002 cts. For the preceding year the average was 1.962 cts. The average revenue per ton per mile was 0.748 ct.; the like average for the year 1905 was 0.766 ct. The earnings per train mile show an increase both for passenger and for freight trains. The figures show an increase in the average cost of running a train 1 mile. The ratio of operating expenses to earnings for the year 1906 was 66.08%. For 1905 this ratio was 66.78%.

EARNINGS AND EXPENSES.—The gross earnings of the railways in the United States from the operation of 222,340.30 miles of line were, for the year ending June 30, 1906, $2,325,765,167, being $243,282,761 greater than for the year 1905. Their operating expenses were $1,536,877,271 or $146,275,119 more than in 1905. The following figures present a statement of gross earnings in detail and show the increase of the several items over those of the previous year: Passenger revenue, $510,032,583—increase, $37,337,851; mail, $47,371,453—increase, $1,945,328; express, $51,010,930—increase, $5,861,775; other earnings from passenger service, $11,314,237—increase, $274,095; freight revenue, $1,640,386,655—increase, $189,613,817; other earnings from freight service, $5,645,222—increase, $564,956; other earnings from operation, including unclassified items, $60,004,087—increase, $7,684,939. Gross earnings from operation per mile of line averaged $10,460, the corresponding average for the year 1905 being $862 less.

The operating expenses assigned to the four general classes were: For maintenance of way and structures, $311,720,820; maintenance of equipment, $328,554,658; conducting transportation, $836,202,707; general expenses, $59,752,230; undistributed, $646,856. Operating expenses averaged $6,912 per mile of line, this average showing an increase of $503 per mile in comparison with the year 1905.

The income from operation or the net earnings of the railways amounted to $788,887,896. This amount exceeds the corresponding one for the previous year by $[illegible],007,642. The net earnings per mile of line for 1906 averaged $3,548; for 1905, $3,189, and for 1904, $2,998.

RAILWAY ACCIDENTS.—The total number of casualties to persons on the railways for the year ending June 30, 1906, was 108,324, of which 10,618 represented the number of persons killed and 97,706 the number injured. Casualties occurred among three general classes of railway employees, as follows: Trainmen, 2,310 killed and 34,989 injured; switch tenders, crossing tenders and watchmen, 147 killed, 1,026 injured; other employees, 1,472 killed, 40,686 injured. The casualties to employees coupling and uncoupling cars were: Employees killed, 298; injured, 3,884.

The number of passengers killed in the course of the year 1906 was 359 and the number injured 10,[illegible]. In the previous year 537 passengers were killed and 10,457 injured. There were 146 passengers killed and 6,053 injured because of collisions and derailments. The total number of persons other than employees and passengers killed was 6,330; injured, 10,241. These figures include the casualties to persons trespassing, of whom 5,[illegible] were killed and 5,927 were injured. The ratios of casualties indicate that 1 employee in every 387 was killed and 1 employee in every 20 was injured. With regard to trainmen it appears that 1 trainman was killed for every 124 employed and 1 was injured for every 8 employed.

In 1906, 1 passenger was killed for every 2,227,611 carried, and 1 injured for every 74,276 carried. For 1905 the figures show that 1,375,856 passengers were carried for 1 killed, and 70,655 passengers were carried for 1 injured. For 1895, 1 passenger was killed for every 2,984,832 carried, and 1 injured for every 213,651 carried. With respect to the number of miles traveled, the figures for 1906 show that 70,126,686 passenger-miles were accomplished for each passenger killed, and 2,338,830 passenger-miles for each passenger injured. For 1905 the figures were 44,320,576 passenger-miles for each passenger killed, and 2,276,002 passenger miles for each passenger injured. The figures for 1895 showed that 71,696,743 passenger-miles were accomplished for each passenger killed, and 5,131,977 passenger-miles for each passenger injured.

A POSITIVELY CONTROLLED, HIGH-SPEED ELECTRIC ELEVATOR.

It is a more or less obvious proposition that the friction brake, as an element of control in the operation of high-speed electric elevators, introduces a danger factor by no means negligible. No friction gear or brake, whatever be the mechanism in which it may figure or the function which it may perform, is positive in its operation. And surely the high-speed elevators in modern high office buildings, with the lives that depend daily upon their safe operation, demand positive control as much as, if not more than, any other mechanism in use to-day.

The excessively high buildings which are growing numerous in the larger cities—particularly New York—are not a suitable field for the hydraulic elevator. The fact that, in the selection of elevators for New York's highest three buildings by three separate, independent boards of engineers appointed for the purpose, the electric traction elevator was adopted in preference to all types of hydraulic elevators, is fairly conclusive evidence in favor of the former. It must be borne in mind that for the upper floors of buildings of such a height high-speed express service must be provided.

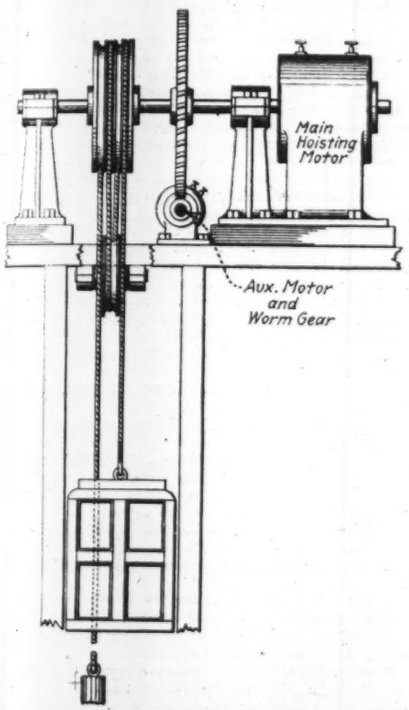
A friction brake, on a slow-speed electric elevator driven by a worm gear of such low pitch that the weight of the car—in the event of failure of the current supply—cannot drive the worm, and hence the motor, is within the limits of its field of safe application. It simply determines a definite stopping-point, and its failure involves no danger to the car. But when an elevator car hangs suspended from a drum direct-connected to the armature shaft of a motor, with no intervening transmission gear, something more positive than a friction brake should be provided to prevent the car from falling in the event of failure of the current supply.

Aside from the question of safety alone, a friction brake involves loss of efficiency, in high-speed elevator operation, for two reasons: (1) In starting the car, before the brake can be released, the motor must be given current enough to overcome the net gravity load and the inertia, plus the friction of the brake; this makes the necessary starting current excessive and de-

lays the start. (2) For lowering the car slowly an armature shunt is required, as it is not practicable to obtain the slowest downward speed required with full field; current is accordingly wasted at a time when the motor, driven as a generator by the descending car, should be delivering current back to the line.

Because of its low efficiency in transmission, and of heating under such circumstances, a low-pitch worm—one which cannot be driven by its gear—cannot be used to drive an elevator car at the high speeds previously mentioned. Heretofore there has been no means of transmission, by which an electric motor could drive an elevator car at high speed, in which the car could not easily drive the motor, with consequent risk of attaining dangerous speed.

Mr. Chas. Richardson Pratt,* M. Am. Soc. M. E., has developed and patented an elevator mechanism which seems to be the first electrically-driven high-speed elevator in which absolutely positive control is obtained. The general



Mechanism of Pratt Positively-Controlled High-Speed Electric Elevator.

principle of this mechanism is shown in the accompanying drawing.

The main shaft is direct-driven by the armature of the large electric motor. The cables (attached to the car at one end and to the counterweight at the other) pass twice over the grooved drum on the main shaft, and over the idler pulley below. Upon the main shaft there is also fixed a large gear, which engages with a low-pitch worm on the shaft at right angles to the main shaft. This worm is direct-driven by a second, smaller electric motor.

The main motor is of sufficient power to take the gravity load. The speed of the small motor is such as to rotate the worm at a rate which will permit the main or hoisting motor to run at suitable speed for the required operation of the car. Being of such low pitch, the worm cannot be driven by the gear on the main shaft. The small motor need have only a tenth or less the power of the main motor.

The motors are so connected that they are controlled simultaneously from the same point—the controller in the car, with its associated rheostats. They may be connected either in series or in parallel, but Mr. Pratt advises us that the former method is expected to be the more satisfactory.

With the elevator car down, the operation is as follows: Upon closing the circuit, both motors start together. The main motor takes all gravity load; and the small one, being of high speed, has a uniform speed acceleration. The worm gear therefore prevents any jerking of the car at starting, such as that ordinarily produced when slow-speed motors are used to drive the hoisting shaft direct.

It is evident that the rotating worm will keep the car at a fixed speed, and that it transmits to the hoisting apparatus a motion which synchronizes with all normal action of the elevator and resists all abnormal action. By normal action is meant a uniform acceleration, after starting, to a given speed; the maintenance of that speed; a uniform retardation to a state of rest; and positive holding when at rest.

Suitable means may be provided, with control located in the car, for varying the speed of the worm-gear motor. By this means the descent of the car at slow speed may be retarded by the worm, thus enabling saving of current in the main motor.

It will be evident from the foregoing description that the worm, by its low pitch, will prevent accidents involving sudden dropping of the car. The car rests at all times, so to speak, upon a low-angle wedge (the worm being merely a wedge wound on a cylinder), which follows it as it is raised or lowered by the main motor.

The load on the worm gear, when in motion, is little or nothing; consequently, the wear upon it is small.

The question may be asked: In the event of sudden stoppage of the worm-gear motor from any cause, with the consequent sudden stoppage of a heavy moving system—loaded car and counterweight—will not the momentum of the latter cause serious trouble? The answer to this is that the momentum of the car and counterweight will overcome the traction of the cables, which will slip around the traction drum until the momentum is gradually arrested. Such stoppage of the small motor, however, is unlikely.

For twelve years there have been in operation on the Central London Railway (underground) 49 elevators, each driven by two motors connected in series and controlled simultaneously from the car. The great size of the cars made it impracticable to drive them with a single worm and gear, so a gear was mounted on each end of the hoisting shaft and a 50-HP. motor and worm installed to drive each gear—the motors series-connected, as explained. This, while not partaking of the features of Mr. Pratt's invention, demonstrates the fact that a combination of two motors simultaneously controlled and positively geared to each other is fully practicable. These elevators and their mechanism were installed under the direction of Messrs. Pratt and Frank J. Sprague.

ANNUAL CONVENTION OF THE CENTRAL STATES WATER-WORKS ASSOCIATION.

The eleventh annual convention of the Central States Water-Works Association was held at Wheeling, W. Va., Sept. 17 and 19. A half-dozen papers were presented, some of which gave rise to considerable interesting discussion. Pittsburg, Pa., was chosen as the next meeting place. Mr. John Langan, of Tipton, Ind., was elected President for the ensuing year, and Mr. Wm. Allen Veach, of Newark, O., was re-elected as Secretary.

In his address of welcome, Mr. C. C. Schmidt, Mayor of Wheeling, stated that the city was about to issue bonds for the improvement of its water supply and that it was undecided whether to build a new reservoir, a sedimentation basin or a filtration plant; and there was also a question as to whether additional pumping capacity of 20,000,000 gals. had best be provided in one or two units. As usual on such occasions, the Mayor expressed the hope that the water-works men present would throw light upon local questions.

COLLAPSE OF A WOOD-STAVE PIPE.

Mr. Latta recounted an unfortunate experience with a 48-in. wood-stave pipe. A total of 2,400 ft. was to be laid, but the pipe collapsed when 600 ft. were in place and at 800 to 900 ft. the scheme was abandoned. The staves were 1⅞ × 4 ins., and the ⅝-in. round iron bands were spaced 16 ins. c. to c. After back-filling 600 ft. of trench a portion of the pipe collapsed. Repairs were made and the bands were spaced 8 ins. c. to c. The pipe again collapsed, and the spacing was narrowed to 4 ins. Signs of collapse appearing again, a 6-in. concrete arch was laid over the wood pipe already built, and for the balance of the 2,400 ft. ⅜-in. steel lock-bar pipe of the same diameter (48 ins.), with flange joints, was laid on a 2 × 6-in. oak grillage. No further trouble has been experienced. The depth of fill over the pipe was not stated. The discussion brought out the fact that the pipe did not collapse so long as it was kept filled with water.

REPAIRING FOOT VALVES.—In a brief experience paper, Mr. C. W. Wiles, of Delaware, O., told how he repaired two flanged, vertical foot valves on 14-in. suction lines, the valves being under 4 to 12 ft. of water. Working on a swinging platform, each foot valve was unbolted. Subsequently the valves were raised to the surface. Mr. Wiles' further description of the work, beginning with the valve in the worst condition, is as follows:

> An examination showed that of the 14 gates 7 or 8 were stuck and covered with mud, so that only about one-half were taking water, thus throttling the supply to the pump, and not seating, allowing the suction to drain when the pump was not in service.
>
> This foot valve was thoroughly cleaned, the valves sandpapered so as to seat, and as all the springs were in good order, in a short time it was placed back, and the pump ran as good as new, taking and discharging easily at all speeds, and with no loss of water in the suction line when the pump was not working.
>
> In view of the results of this examination it was thought best while the tackle was in place to examine the other foot valve, though the pump seemed to work fairly well. After disconnecting and hoisting out this second foot valve, it was found to have several gates out of order and the bottom plate on which the nest of valves rested was found to be broken entirely across one way and one-half across the other.
>
> Six of the 14 valves were stuck, and the only way the pump could take sufficient water was by the broken bottom plate opening up, allowing the water to pass through when the pump was working, and closing down when the pump was stopped. It was found that many bolts were badly worn by the movement of the broken plate. A new bottom, including gates and screen, was placed on the old top, and all replaced in the well.
>
> Since cleaning and repairing these foot valves, the pumps have worked finely, running with less power and at a saving of steam.

MUNICIPAL OWNERSHIP.—A lengthy paper on this subject was sent in by Mr. John W. Hill, M. Am. Soc. C. E., of Cincinnati. The author expressed himself strongly against municipal ownership. He stated as his belief "that public officials and contractors, in point of honesty and integrity, will compare favorably with any other class of men." Yet, owing to political and other conditions, he was of the opinion that municipal public works could not be built and operated as cheaply or as efficiently under public as under private ownership. Personal experiences [some of which were apparently connected with the Philadelphia filtration works] were cited in support of this belief. Many quotations from various writers were made and commented on as further support of the author's thesis.

Mr. A. L. Holmes, of Grand Rapids, Mich., expressed general agreement with Mr. Hill, and Mr. Latta cited a supporting incident, consisting of a ridiculous system of municipal bookkeeping by wards, to no purpose whatever, except to comply with an unwise statutory requirement. Mr. Jerry O'Shaughnessy, of Columbus, objected strongly to some statements in Mr. Hill's paper reflecting on the conduct of the municipal electric lighting plant at Columbus.

"WATER, WATER, EVERYWHERE, AND NOT A DROP TO DRINK."—Dr. S. L. Jepson, of Wheeling, made use of this poetical text for a lengthy but well-written sermon on the importance of pure water supplies in general and the absence of such at Wheeling in particular. He cited, as perhaps the first recorded instance of a typhoid outbreak traced to drinking water, an experience at North Boston, related by Dr. Austin Flint, Sr. This hamlet, near Buffalo, contained in 1843 a tavern and eight dwellings, occupied by 43 people. Typhoid, it is said, was un-

*Consulting Mechanical Engineer, 1123 Broadway, New York City.

known in the county until a stage passenger suffering from the disease alighted at the tavern and died there. Six of the nine families in the hamlet used water from the tavern pump, and all the members (28) of these families contracted the disease. The three families using different water escaped.

What the author rightly characterized as "a startling exhibit" of the need of an improved water supply on the part of Wheeling was presented in tabular form. For the 20 years from 1884 to 1903, the average typhoid death rate in Wheeling has been 125 per 100,000. The first 10 years of this period, prior to the erection of a new pumping station at a more favorable locality, gave a death rate of 179.63, against 72.35 for the following 10 years. For 5-year periods, the typhoid rate, beginning with 1884, has been 125, 231, 54.5, and 90; the latter of course showing a recent marked deterioration in the quality of the supply.

Mr. Fred Leopold, in the course of a discussion of the foregoing paper, spoke of recent reductions in the price of sulphate of alumina of some 25%. He also stated that at new stock yards in Chicago a 5,000,000-gal. filter plant is being built "to purify the waters of Bubbly Creek, which carries all the drainage and discharge from the slaughter-houses." An experimental plant was operated for several months. Sulphate of iron, combined with a small percentage of copper, will be used.

"NATURAL FILTRATION" AT OWENSBORO, KY.—Mr. Ferdinand Grote ["Smith-Grote Slow Sand Filtration System"] in a paper entitled "Sand Filtration by Nature's Method," and Mr. Asa Williams, Chief Engineer of the Water-Works of Owensboro, Ky., in the course of the discussion on Dr. Jepson's paper, described an interesting new water supply recently secured for Owensboro. This consists of 36 Cook strainer tubes 16 ins. in diameter and 16 ft. long, laid in a sandbar in the Ohio River. The strainers have fine slots or waterways and are buried in 4 ft. of sand. On a test, water was pumped through the strainers at the rate of 5,000,000 gals per day. Mr. Grote stated that the flow of water in the river keeps not only the surface but the whole body of the sandbar washed clean. Analyses make by the State Board of Health of Ohio showed considerable numbers of B. coli in the river water, but none in the water drawn from the sandbar. The "filtered water" showed considerable algal growth similar to crenothrix, suggesting the desirability of watching the growth and the possible necessity of treating "the strainer pipes by back flushing with water containing copper sulphate."

WATER METERS.—Mr. A. L. Holmes, of Grand Rapids, Mich., presented a paper in favor of metering. He stated that one of the most serious defects in water-works operation is pump slippage, which often runs from 25 to 50%. He said that most small water-works have not means of testing the output of their pumps, and spoke of the usefulness of the pitometer for this purpose. All water should be measured as it leaves the pumping station, and again measured as delivered to consumers, in order that the one may be checked against the other and, in case of material discrepancy, means taken to stop leaks. The fallacy of basing water rates in one town upon those in another was condemned, and it was urged that each works "should stand upon its own merits, and the cost of construction, the supply cost," and various other pertinent factors, be used as a basis for water rates.

Mr. John Langan, of Tipton, Ind., read a paper dealing with the advantages of deep wells as sources of water supply. At Tipton water is drawn from wells 60 to 440 ft. deep, by means of air lifts. The air introduces aids in the deposition of iron and other minerals in the reservoir.

OTHER PAPERS.—A paper on garbage disposal was read by Mr. Thomas R. Cooke, of Toledo, O., and gave rise to a number of inquiries and considerable discussion. An interesting discussion on "The Best Method of Unjointing Cast-Iron Pipe" will be given space in a later issue.

Talks accompanied by lantern-slide views were given during the convention by Messrs. Fred Leopold and Morris Knowles, M. Am. Soc. C. E., both of Pittsburg, the former taking up mechanical and the latter slow-sand filtration.

A MINE EXPLOSION at Rock Springs, Wyo., on Sept. 12, injured 40 men. Miners' lamps ignited coal dust that had been stirred up by a runaway coal car.

Eleven miners were killed on Sept. 20, at Negaunee, Mich., by the breaking of a hoisting cable and dropping of the cage in the shaft of the Jones & Laughlin Steel Co.'s mine. Seven other men on the cage were fatally injured.

At the Wharton mines, Upper Hibernia, N. J., a cage bearing seven men was dropped 250 ft. on Sept. 23, and two of the men probably fatally injured. The engineer fainted while the cage was descending, and it was dashed to the bottom of the shaft.

SINKING OF LAND on Sept. 23, near Creston, Ohio, engulfed a locomotive and four cars on the new 11-mile cut-off of the Baltimore & Ohio R. R. from Creston to Sullivan; the train crew narrowly escaped by jumping. At nearly the same instant a stretch of the county highway, near that point, sank 15 ft. The cause is believed to have been failure of the crust over an underground lake, the existence of which has been suspected for years.

AN EXPLOSION wrecked the generating plant of the gas works at Perth Amboy, N. J., on the afternoon of Sept. 23. One man was killed and seven injured. The explosion, of unknown origin, caused damage estimated at about $50,000. Besides Perth Amboy, this plant served also the towns of South Amboy, Keasby, Woodbridge, Sewaren and Metuchen; and these other towns may be deprived of gas service until the damage is repaired.

THE MOST SERIOUS WRECK of the past week occurred on the Mexican Central Ry. at Encarnacion, near Guanajuato, on Sept. 19; press reports vary widely as to the number of deaths, from 32 to 63 being given, with 40 or more injured. The wreck was the result of a head-end collision between the El Paso express and a freight train; the engineer of the freight train is said to have confessed to disobedience of orders.

Another wreck on the Lake Shore Electric Ry., on which several bad accidents have lately occurred, killed two and injured 25 on Sept. 19, two miles out of Toledo; one of the fast limited cars ran into an open switch, an automatic one which, it is reported, failed to work; the telephone system of dispatching is used on the road.

Derailment of a Southern Ry. limited train on Sept. 22, near Charlottesville, Va., injured 32 passengers. A broken rail is said to have caused the accident. The train took fire and three sleepers were entirely burned.

A freight wreck occurred on the Atchison, Topeka & Santa Fe Ry. on Sept. 21, in which five men were killed and one fatally injured. A long train got away on a grade and struck a light engine at the bottom; fire destroyed a part of the wreckage.

A Lehigh Valley express left the track on Sept. 17 at the entrance to the Musconetcong tunnel, near Easton, Pa. Seven persons were injured. The train, leaving the rails, plunged against the side of the mountain beside the tunnel portal.

THE LARGEST STEAMBOAT yet built for the Hudson River service was launched at Camden, N. J., on Sept. 21. The boat is the "Princeton," built by the New York Shipbuilding Co. for the Peoples' Line. She is 440 ft. long, 95 ft. beam, and has four decks. There will be 500 staterooms, three tiers of which will open directly on the main saloon; altogether the vessel will have sleeping accommodations for 2,000 persons.

FIVE TURBINE-DRIVEN DESTROYERS are to be built for the navy. Secretary Metcalf awarded the contracts on Sept. 20; bids were opened about a month ago. The successful bidders were: The Wm. Cramp & Sons Ship & Engine Building Co., two vessels at $585,000 each; the Bath Iron Works, two vessels at $624,000 each; and the New York Shipbuilding Co., one ship at $645,000. The hulls will be built from Navy Department plans, and the machinery from the bidders' plans. Parsons turbines will be used.

THE "MAURETANIA," sister-ship of the "Lusitania," and the largest vessel ever built on the Tyne, started on her preliminary sea trials on Sept. 17. This trial was what is known as the builders' trial, and was to be carried out between Flamborough Head and the Aberdeenshire Coast, the vessel anchoring off the Tyne each evening. The construction of the "Mauretania" was begun in 1904, by Messrs. Swan, Hunter and Wigham Richardson—already represented in the Cunard fleet by the "Ultonia," the "Carpathia" and the "Ivernia." Substantially her dimensions are the same as those of the "Lusitania," though in some of the figures she is a trifle the larger.

AN AMERICAN PEAT ASSOCIATION is proposed for the organization of persons interested in the development of American peat lands and in the production of peat fuels. A meeting for completing organization will be held at the Jamestown Exposition, Va., Oct. 15 to 20. A voluntary committee of 23 is making arrangements for the meeting and the organization. The chairman is Dr. Joseph Hyde Pratt, Mines and Metallurgy Building, Jamestown Exposition, Va., and Julius Bordollo, Kingsbridge, N. Y. City, is Temporary Secretary-Treasurer.

THE FIRST THROUGH CAR under the East River at New York City was run from Long Island City to Lexington Ave. and 42d St., Manhattan, through the north tube of the so-called Steinway or Belmont tunnel on Sept. 23, 1907. This marks the first tangible result of the various tunnel systems which will connect Manhattan Island to the New Jersey mainland and to Long Island. The Steinway or Belmont tunnel has an interesting history. It is a twin tube steel frame and concrete-lined tunnel running from grade in Long Island City under the East River, through East 42d St., Manhattan, to the Grand Central Station, reaching the latter at a much lower grade than any of the subsurface work now existing there. The present western terminus is at present at Lexington Ave., some two blocks east of the station, where it comes to a dead end with no connections to any of the city subways or tunnels. The work was started in July, 1905, by the local traction interests, working on an old franchise granted many years ago, the legality of which is now under discussion in a suit instituted by the city against the builders. This old franchise allowed no connections in the city, hence the present terminal conditions. The car, operating on this first through trip, is of steel construction throughout and operated by a short-pole trolley connection to overhead rails. It is one of the series of cars to be used in six-car multiple-unit trains. The future use of the tunnel and the problem of its control are matters of the deepest secrecy, which will probably not be decided until the outcome of the city's suits can be known.

THE SECOND EXPOSITION OF ELECTRICAL APpliances of the Agricultural, Scientific and Industrial Society of Lyons, France, is announced for May, 1908. The first exposition was held at Lyons in May, 1906, with exhibits principally by Frenchmen. It is desired that the 1908 exposition should include a large and comprehensive display of American electrical apparatus. It is stated that British, Italian and German manufacturers will display their lines. A circular issued by the above-mentioned society classes the appliances as follows:

First Class: Electrical apparatus for farming, dairying, irrigating and pumping.

Second Class: Appliances for textile industry.

Third Class: Tools for wood and metal working, cranes, switches, couplers, mining machinery, traction apparatus, domestic mechanical appliances.

Fourth Class: Electric lighting material.

Fifth Class: Domestic and industrial heating appliances; electrical refrigeration apparatus; electrochemical equipment.

Sixth Class: Apparatus for the generation, transformation and transmission of electricity.

Seventh Class: Telephones, alarms, calls, signals, medical instruments, miscellaneous small apparatus.

Exhibits of an industrial nature for any useful channel will be welcomed, though not directly included in the above seven classes.

STREET RAILWAYS with cars operated by manual power are in use at Mombasa, in East Africa. The light, narrow-gage tracks are laid through the street, and in the unique system of operation the cars are for hire, like cabs, or are the private property of officials and wealthy residents. They are little four-wheel cars with one or two cross seats, and each is propelled by two natives. Spur tracks are run into private grounds, so that persons can take the cars to their doors.

THE RAILWAYS OF NEW SOUTH WALES, Australia, aggregated 3,453 miles open for traffic on June 30, 1906; 145 miles were under construction and 360 miles authorized. The gross earnings were $23,547,000, with operating expenses $12,498,700 (53%). During the year 41,413,084 passengers were carried, without any train accident resulting in loss of life. A large amount of work was done in laying new rails, ties and ballast. At the railway shops 37 locomotive boilers were built and 40 locomotives rebuilt, while 724 engines were repaired. The freight traffic included 8,542,850 tons of general freight and 250,980 tons of live stock. The equipment comprises 656 locomotives, 541 tenders, 1,187 cars in passenger service, 11,681 cars in freight service, and 1,038 cars in railway service. The maintenance of way and structures represented $865 per mile, 12.6% of reve-

nue and 29.73% of operating expenses. The locomotive and car maintenance and repairs represented $1,650 per mile, 24.9% of revenue and 45.3% of operating expenses. The street and interurban railways aggregated 130 miles, some operated by steam locomotives and others by the overhead electric system. There were 5 miles under construction and 12 miles authorized. The total earnings were $4,543,500, and the operating expenses $3,639,700 (80.11%). The equipment comprises 60 motors, 76 steam cars, 727 electric cars and 43 service cars. The employees number 15,180 on the railways and 4,334 on the street railways. All these lines are owned and operated by the government. Mr. T. R. Johnson is Chief Commissioner of Railways; Mr. James Fraser, Engineer in Chief for Existing Lines; Mr. W. Thow, Chief Mechanical Engineer; Mr. G. R. Cowdery, Engineer for Street Railways, and Mr. O. W. Brain, Electrical Engineer.

A TRAIN-FERRY STEAMER, the "Lucia Carbo," has been built for the Entre Rios Railway, of the Argentine Republic. It will have a run of about 50 miles on the River Plate to connect this road with lines running to the port of Buenos Ayres. It is of steel construction, 278 ft. long, 56-ft. beam and 18 ft. 10 ins. deep. At the middle are steel columns supporting an upper deck with superstructures for pilot house, quarters and cabins. There are three tracks, one in the center and the others curving along the sides of the deck. The twin screws are driven by triple-expansion engines, steam being supplied by four single-ended Scotch boilers carrying 175 lbs. pressure. The ship was built and engined by A. & J. Inglis, of Glasgow, Scotland. On the trial trip it made 14½ knots with the engines running at 150 r. p. m. and developing 2,250 HP. The ship went out from England under its own steam, carrying sufficient coal for the entire voyage, which was made in about four weeks.

SEWAGE DISPOSAL FOR SEASIDE TOWNS is the subject of a paper recently presented before the Royal Sanitary Institute of Great Britain by J. T. C. Nash, M. D., Medical Officer of Health, Southend-on-Sea. In opening his paper, Dr. Nash made the following quotation from one of the recent reports of the Royal Commission on Sewage Disposal:

> The treatment of sewage, according to methods at present in use, cannot be relied on so to alter its character as to allow of its discharge in the immediate neighborhood of shell-fish layings without incurring appreciable risk of disease being communicated by the consumption of shell-fish taken from such layings.

This statement was used by Dr. Nash in justification of the opinion that there are "no very serious objections to the present methods of disposal by discharge into the sea which are not also applicable to treated sewage, but there are some minor ones chiefly of an esthetic or sentimental nature."

The paper concluded (with some omissions) as follows:

> The method of disposal of sewage that would seem to be sufficient for a sea-coast town would be (1) screening off of grosser solids; (2) precipitation and liquefaction in tanks of sufficient size to hold a day's dry weather flow. This would permit of partial anaerobic bacterial treatment and consequent breaking down of the contained organic matter; (3) the effluent to be discharged at an outfall sufficiently far from the shore. In no case should the outfall terminate on the foreshore. It should always be covered with water, even at low water of spring-tides.
>
> The sea in itself constitutes better oxidizing contact beds than any expensive artificial contact beds, but sentiment if nothing else calls for some preliminary reduction of impurity prior to discharging sewage even into the sea.
>
> Among the unsolved problems in the disposal of the sewage of large towns, are the questions of overflow and of storm water in times of rain. If ever the time comes when the horse and the dog are tabooed in cities, the storm water problem will have shrunk into comparative insignificance; but as long as horse traffic predominates, the first washings of streets in times of storm, after a period of dry weather, must be indescribably filthy. In my judgment the first washings of streets should always reach the main sewers.

PERSONALS.

Mr. Geo. P. Graham, of Brockville, Ont., has been appointed Minister of Railways of Canada.

Mr. Daniel R. Norton has been appointed Manager of the Security Steel & Iron Co., of Troy, N. Y.

Mr. C. C. Stroufe was recently appointed Assistant to the Chief Engineer of the Oregon Short Line Ry., with offices at Portland.

Mr. J. F. Deimling has been appointed Chief Engineer of the Pere Marquette Ry., succeeding Mr. E. K. Woodward, who has resigned.

Mr. T. E. Layden, Assistant Engineer of Tests on the Atchison, Topeka & Santa Fe Ry., has been appointed Engineer of Tests, with offices at Topeka, Kan. He succeeds Mr. F. W. Thomas.

Mr. Jas. F. McLaughlin has been appointed Chief of the Electrical Bureau of Philadelphia, succeeding Mr. John Sager, who is resigning on account of his health. The change will take place Oct. 1.

Mr. Harry J. Kesner has been appointed Instructor in Bridge Engineering in the University of Minnesota. Mr. Kesner is a graduate from the Civil Engineering Department of the University of Colorado.

Dr. Thos. T. Read, M. Am. Inst. M. E., Professor of Mining and Metallurgy in Colorado College, at Colorado Springs, will go to China to occupy the chair of Metallurgy at the Imperial University of Tientsin.

Mr. Edward E. Goudey, who was formerly connected with the Bell Engineering Co., of New York City, is now Western Manager of the Wallace-Coates Engineering Co., with offices in the Sherlock Building, Portland, Ore.

Mr. J. E. A. Moore, M. Am. Soc. C. E., has resigned his position with the Wellman-Seaver-Morgan Co., of Cleveland, to associate himself with Mr. John W. Seaver, M. Am. Soc. C. E., in general consulting and contracting engineering business.

Mr. H. H. McGee has been appointed Engineer in Charge of New Construction and Maintenance of Bridges, Buildings and Structures for the Ulster & Delaware R. R., reporting to the General Superintendent. Mr. L. E. Powers has been appointed Roadmaster.

Mr. Geo. P. Dieckmann has been appointed Chief Chemist for the Northwestern States Portland Cement Co. (Cowham System) at Mason City, Iowa. He was formerly connected with the Peninsular & Empire Portland Cement Co.

Mr. Howard C. Ford, C. E., has been appointed Assistant Professor of Irrigation Engineering and Surveying at Iowa State College, Ames, Iowa. Mr. Ford is a graduate of the University of Colorado and has been Instructor in Civil Engineering in that institution for three years.

Mr. W. G. Bierd, for two years General Manager of the Panama R. R., has tendered his resignation, to take effect Oct. 1. He goes to the New York, New Haven & Hartford R. R., where he will succeed Mr. O. M. Shepard as General Superintendent. Mr. Shepard will be assigned to other duties.

Messrs. Clement C. Williams and Arthur H. Poorman have been appointed Instructors in Civil Engineering in the University of Colorado. Mr. Williams is a graduate of the University of Illinois and has had nearly two years' experience with the Delaware, Lackawanna & Western R. R.; Mr. Poorman graduated from the University of Illinois in 1907 with special honors and has since been in the employment of the Weber Concrete Construction Co.

Obituary.

David B. Reeve, Chief Engineer of the New England Steamship Co., died on Sept. 12. He was 73 years of age.

Ferman J. Stout, General Manager of the Lake Shore Eletcric Ry., died at Toledo on Sept. 15. He was born at Deerfield, Mich., in 1858.

H. W. Hill, of Cleveland, founder of the Hill Clutch Co., and formerly engaged in general engineering work, died at Cleveland on Sept. 7. He was 54 years old.

A. M. Mullinix, of Ft. Worth, Tex., Superintendent of Bridges and Construction for the Ft. Worth, Denver & Gulf Ry., died in Denver on Sept. 14 at the age of 49.

Henry Clarkson Wicker, formerly President of the Ft. Worth & Rio Grande Ry., died on Sept. 20 at his home at Glen Cove, L. I. He was born at North Ferrisburgh, Vt., in 1840.

Daniel Bacon, Manager of the Marine Magnetic Control Co., of New York City, was killed by a sight-seeing automobile on Sept. 15, while crossing Park Ave. He was 54 years old and unmarried.

M. Dewitt Woodford, former President of the Cincinnati, Hamilton & Dayton R. R., died on Sept. 15 at Kalamazoo. He was born at Fredonia, N. Y., in 1839, and was one of the best known railroad men in the West.

Col. John E. Greer, Ordnance Department, U. S. A., Chief Ordnance Officer of the Department of the East, died on Sept. 19 at his home on Governor's Island. He was born in 1844, and graduated from West Point in 1867. Since his graduation he had devoted his services to ordnance work.

Edwin Morey, a former President of the Boston & Lowell R. R., died of heart failure in Boston on Sept. 21. He had just passed his 80th birthday. While he was President of that road he appointed as his General Passenger Agent Mr. Chas. S. Mellen, now President of the New York, New Haven & Hartford R. R.

Dr. Geo. Archbold, a well-known consulting chemist and formerly Chief Chemist of the War Department, died at his home in Jersey City, N. J., on Sept. 16. He was born in Scotland in 1849. In 1881 he came to America, and in 1890 became Chief Chemist of the War Department; while in that position he built and equipped the naval laboratory at Washington.

George W. Plympton, M. Am. Soc. C. E., Senior Director of Cooper Union, New York City, and for 42 years Professor of Physical Sciences at the Brooklyn Polytechnic Institute, died at Tyson, Vt., on Sept. 11, at the age of 80. In 1847 he was graduated as a civil engineer from the Rensselaer Polytechnic Institute at Troy, N. Y., and taught in that school for a time. He served during the Civil War as an engineer, and as State Surveyor in the Adirondacks, before again taking up teaching.

Samuel Sloan, one of the best-known railroad men in the country, and for 30 years President of the Delaware, Lackawanna & Western R. R., died on Sept. 22 at his home at Garrisons, N. Y. Had he lived until Christmas he would have been 90 years old. He was born at Lisburn, Ireland, on Christmas day, 1817. When two years of age his parents came to this country. Mr. Sloan's railroad career began in 1855, when he became a director, at the instance of Gov. Morgan, of the New York Central—then known as the Hudson River R. R. and being but a single-track road extending to Peekskill. Soon afterwards he became President of the road, and under his administration the stock rose from $17 a share, when he became President, to $140 in 1867, when he retired. Two years later he was elected President of the Delaware, Lackawanna & Western R. R., and in 1899 he was made Chairman of its Board of Directors, a position which he held until his death. On Christmas, 1897, the employees of the Lackawanna gave him, in appreciation of his services, a solid gold tea set costing $100,000. Besides the railroad experience previously mentioned, he had held, at one time or another, the presidency of several other roads, including the Michigan Central R. R.; the Rome, Watertown & Ogdensburg; the International & Great Northern, and the Marquette, Houghton & Ontonagon. At his death Mr. Sloan was a director or officer in 33 corporations, including, besides various railroad companies, the Consolidated Gas Co., the Western Union Telegraph Co., and several banking and trust companies.

Rear-Admiral John Grimes Walker, U. S. N., retired, died at Cape Meddick, Me., on Sept. 16. He was 72 years of age. Admiral Walker was most widely known for his work in connection with the Isthmian Canal enterprise. He was appointed by President McKinley Chairman of the Nicaragua Canal Commission of 1897-'99, and was generally considered to be a strong friend of the Nicaragua route. In 1899 he was again made Chairman of the Isthmian Canal Commission created by Congress to report on the comparative merits of Panama and Nicaragua. When the Panama route was finally adopted and the appointment of a new Commission to have charge of the construction of the canal became necessary, Admiral Walker was again selected to head it. The failure of this Commission to administer the work efficiently is a matter of recent history. To what extent the Chairman was responsible for this failure is a subject of possible disagreement; but there can be little question that a chairman not subject to the physical infirmities inevitable with advancing years might have produced different results.

Admiral Walker was a prominent figure in the Navy for many years. His Civil War record, although made when little more than a youth, was a creditable one. From 1881 to 1889 he was Chief of the Bureau of Navigation. On Oct. 1, 1889, he was appointed Rear-Admiral and took command of the recently built and organized squadron of evolution. In 1890 he conducted the famous European cruise of the squadron.

ENGINEERING SOCIETIES.

COMING MEETINGS.

AMERICAN PUBLIC HEALTH ASSOCIATION.
Sept. 30-Oct. 4. Thirty-fifth annual meeting at Atlantic City, N. J. Secy., Dr. Chas. O. Probst, Columbus, Ohio.

AMERICAN SOCIETY OF MUNICIPAL IMPROVEMENTS.
Oct. 1-4. Annual meeting at Detroit. Secy., Geo. W. Tillson, 831 Ocean Ave., Brooklyn, N. Y.

NATIONAL ASSOCIATION OF RAILWAY COMMISSIONERS.
Oct. 8. Annual meeting at Washington, D. C. Secy., Edw. A. Moseley, Interstate Commerce Commission, Washington, D. C.

RAILWAY SIGNAL ASSOCIATION.
Oct. 8-10. Twelfth annual meeting at Milwaukee. Secy., C. C. Rosenberg, Lehigh Valley R. R., South Bethlehem, Pa.

AMERICAN BOILER MANUFACTURERS' ASSOCIATION.
Oct. 8-10. Nineteenth annual convention at Atlanta, Ga. Secy., J. D. Farasey, Forest St. and Erie Ry., Cleveland, O.

INTERNATIONAL ASSOCIATION OF FIRE ENGINEERS.
Oct. 8-11. Thirty-fifth annual convention at Washington, D. C. Secy., Jas. McFall, Roanoke, Va.

AMERICAN ELECTROCHEMICAL SOCIETY.
Oct. 10-12. Annual fall meeting at New York City. Secy., Prof. J. W. Richards, Bethlehem, Pa.

AMERICAN STREET AND INTERURBAN RAILWAY ASSOCIATION.
Oct. 14-18. Annual convention of this and affiliated association at Atlantic City, N. J. Secy., Bernard V. Swenson, 29 West 39th St., New York City.

AMERICAN STREET AND INTERURBAN RAILWAY ENGINEERING ASSOCIATION.
Oct. 14-16. Annual convention at Atlantic City, N. J. (See preceding item.) Secy., S. W. Mower, Southwestern Traction Co., London, Ont.

ASSOCIATION OF RAILWAY SUPERINTENDENTS OF BRIDGES AND BUILDINGS.
Oct. 15-17. Annual meeting at Milwaukee. Secy., S. F. Patterson, Concord, N. H.

AMERICAN GAS INSTITUTE.
Oct. 16-18. Second annual meeting at Washington, D. C. Secy., Jas. W. Dunbar, New Albany, Ind.

ROADMASTERS' AND MAINTENANCE OF WAY ASSOCIATION.
Nov. 11-14. Twenty-fifth annual convention at Chicago. Secy., W. E. Emery., C. & A. Ry., Kansas City, Mo.

AMERICAN MINING CONGRESS.
Nov. 11-16. Tenth annual convention at Joplin, Mo. Secy., J. F. Callbreath, Jr., Denver.

AMERICAN CIVIC ASSOCIATION.
NATIONAL MUNICIPAL LEAGUE.
Nov. 19-21. Joint convention at Providence, R. I. Secy., Clinton R. Woodruff, North American Building, Philadelphia.

SOCIETY OF NAVAL ARCHITECTS AND MARINE ENGINEERS.
Nov. 22-23. Annual meeting at New York City. Secy., W. J. Baxter, 29 West 39th St., New York City.

WESTERN RAILWAY CLUB.—The first meeting of the season was held Sept. 17 at the Studebaker Fine Arts Building, Chicago; Mr. C. A. Seley was in the chair. The secretary announced that the membership now numbers 1,406. A paper by Mr. W. D'A. Ryan, illuminating engineer, on "Exposed, Concealed and Semi-Concealed Car Lighting," was read, in the absence of the author. It related mainly to a system of car lighting by electricity in which the lamps are fitted to sockets in a wide dish-shaped reflector attached to the ceiling or headlining. The lamps are concealed by an ornamental trough of opalescent glass, and the effect of the reflector is to give a good diffusion of the light. An exhibition of the system was given in a cabinet representing a section of a car.

EMPIRE STATE GAS AND ELECTRIC ASSOCIATION.—The annual convention will be held in the auditorium of the New York Edison Co., 44 West 27th St., New York City, on Oct. 2, at 10 a. m. Papers arranged for presentation are: "The Meter-Testing Situation, and Report of the Meter Committee," R. M. Searle; "Gas Standards," Alfred E. Forstall, M. Am. Soc. M. E.; "Buying Light," E. L. Elliott; "Franchise Taxation," L. T. Palmer; "The Agitation for Underground in Place of Overhead Distribution," W. W. Cole; "Municipal Ownership in New York State," Glenn Marston.

A joint meeting of this Association and the Street Railway Association of the State of New York, and a reception, will be held in the concert hall of Madison Square Garden on Oct. 1, at 8 p. m. Addresses will be made as follows: "The Work of the Public Service Commission and Its Policies with Relation to the Corporations Under Its Supervision," Frank W. Stevens, Chairman of the Commission; "The Electric Railway Situation of To-day," Henry J. Pierce, President, International Ry. Co., Buffalo; "Control of Gas Companies by State Commissions," Dr. A. C. Humphreys, President, Stevens Institute of Technology; "Public Control from the Corporate Standpoint," Everett W. Burdett, Chairman, Committee on Public Policy, National Electric Light Association.

NEW YORK RAILROAD CLUB.—The first meeting of the fall was held on Friday evening, Sept. 20, at the Engineering Societies' Building, 29 West 39th St.; President H. H. Vreeland occupied the chair. The report of the Committee on Nominations was presented, and recommended a continuance of the present staff of officers, whose terms expire Nov. 1. These officers are as follows: President, H. H. Vreeland; Vice-Presidents, John F. Deems (N. Y. C.), W. G. Besler (C. R. R. of N. J.), and H. S. Hayward (P. R. R.); Secretary, H. D. Vought; Treasurer, R. M. Dixon. These candidates will be voted for by letter ballot and the returns announced at the meeting on Nov. 15.

The paper of the evening was "The Steam Locomotive versus the Electric Locomotive," by Mr. Max Toltz, of St. Paul, Vice-President and General Manager of the Manistee & Grand Rapids R. R. In the main, Mr. Toltz's paper was an argument in which he took issue with Messrs. Lewis B. Stilwell and Henry St.C. Putnam in an estimate which they presented before the American Institute of Electrical Engineers in January last, in which they maintained that if all the railways of the United States were to-day operated by electricity the aggregate cost of operation, which in 1905 was $1,400,000,000, would be reduced by about $250,000,000.

Mr. Toltz's argument was strongly in favor of the steam locomotive; and he sought to demonstrate that the steam locomotive, properly improved, is far more economical than the electric locomotive, even on a basis of 1 KW.-hr. of electrical power furnished as cheaply as 0.6 ct. at the bus-bars, or 0.8 ct. effective for traction, as was maintained by Messrs. Stilwell and Putnam. He discussed fully the value of superheaters on steam locomotives, and strongly recommended their use; with superheaters, smoke consumers, and automatic stokers, the work of engine crews would be diminished greatly: firemen would handle less coal, and the use of superheated steam would make far better lubrication in the cylinders.

The Interstate Commerce Commission reported 47,000 steam locomotives in service in 1904, aggregating about 16,625,000,000 HP. Assuming that these three improvements can be added at a cost of less than $4,000 a locomotive, Mr. Toltz figured that the total expenditure for 47,000 locomotives would be $188,000,000 and this would be repaid within 22 months and 12.5 months, respectively, according to the following propositions:

Basing calculations upon the existing types of locomotives in 1904, Mr. Toltz shows that a total saving of $103,243,000 per year can be effected. Of the 47,000 locomotives mentioned, however, about 60% were built over 15 years ago; the foregoing figure was based on an average efficiency computed from full-consumption figures, etc., forming a part of the 1904 report—the average fuel consumption being about 4¼ lbs. per draw-bar HP.-hr. Assuming all locomotives in this country to be of the latest types, using only 2.6 lbs. of coal per draw-bar HP.-hr., their equipment with the three economizing devices would produce a saving of $177,880,000.

According to Mr. Toltz, Messrs. Stilwell and Putnam maintained that the complete electrification of existing steam roads would bring about a corresponding saving of $232,758,000. In view of the huge expenditure of capital—conservatively estimated at several billions of dollars—necessary to a complete electrification, with by no means certain results as to the amount to be saved, and of the smaller expenditure (estimated, as mentioned previously, at $188,000,000) necessary to equip existing steam locomotives with economic devices which will effect a saving nearly as great as that estimated by Stilwell and Putnam, Mr. Toltz stood strongly in favor of the steam locomotive for trunk-line service—reserving the electric locomotive for service at large terminals and in tunnels.

WESTERN SOCIETY OF ENGINEERS.—At the meeting held at the Society's rooms in Chicago on Sept. 18, Mr. H. Kreisinger presented a paper written jointly by himself and Mr. W. T. Ray on "The Nature of True Boiler Efficiency." It was in the main an abstract of a bulletin, to be issued by the steam engineering division of the U. S. Geological Survey, and dealt mainly with laboratory experiments made to determine the laws which govern heat absorption by boilers. True boiler efficiency is defined as the ratio of the heat absorbed by the boiler to the heat which is available for it, counting only that heat as available which is above the temperature of the boiler water. Small multitubular boilers were used in the tests, as they are of the simplest form. Water-tube boilers present more complicated problems on account of the irregular and variable passages for the gases. The flues were thin copper tubes in order to reduce the resistance of the metal to a minimum, and the air was heated in an electric furnace to avoid the formation and deposition of soot. At temperatures of 1,200° to 1,500° F. the nickel wire of the furnace became very brittle after cooling, and failed by fusing after being in use 10 hours a day for 6 to 10 days. The deductions drawn from the experiments were summarized as follows:

1. Beyond the points of the critical velocity the rate of heat absorption by convection by the heating surface of boiler flues is nearly a straight line function of the velocity of the gas. 2. The deviation from the straight line function becomes larger with the rise of the initial temperature. 3. The critical velocity rises with the initial temperature of the gases, and drops when the diameter of the flues increases. 4. The rate of heat absorption increases when the initial temperature rises; it also seems to vary directly with the density of the gas. 5. Increasing the diameter of flues decreases their efficiency as heat absorbs, increasing the length of flues beyond a certain length increases their efficiency very little. 6. Most of the resistance to the passage of air through the flues is at the entrance into the tubes, the length of the flues increases the resistance but little.

The paper was followed by an extended discussion, relating mainly to the nature of the experiments and to the application of the deductions in practical work. In a written discussion, Mr. Ray suggested that it might well be claimed that the actual temperature of the metal of the fire tubes should have been carefully determined, since that temperature (and not the temperature of the water) is the lower limiting temperature of cooling the gases. The difference in the temperatures of the metal and the water is of vital importance. In view of the comparatively low temperatures to be used in the earlier tests, however, it was concluded that the difference would not be sufficient to affect the results. With high furnace temperatures, the true boiler efficiency fell gradually with increasing heat transmission. This, he believed, indicated an increasing rise in the temperature of the tube metal, due to the water circulation failing to keep up with the demands. There was also difficulty in making determinations of tube-metal temperatures in small laboratory boilers, but this will be done with the large boilers in the testing plant now installed at Norfolk.

As to the effect of draft, this is used up in two ways: (1) By the desired scrubbing of the heat surface; (2) by contraction and expansion of the cross-section of flow; the loss due to this is accompanied by scarcely any benefit. If boilers are to be worked much harder, useless absorption of draft must be greatly reduced by intelligent arrangement of baffles and heating surfaces. Mr. Ray believed that few forced and induced-draft fans have a high efficiency, because the air that enter and leave the blades with considerable shock as the blades are usually radial. High efficiency in fans of this kind will be absolutely necessary for high drafts if boilers are to be made to absorb ten times the present per square foot of heating surface. In attempting to force up steam output per square foot of heating surface, per square foot of floor space, and per dollar invested, the greatest troubles will be with the fuel bed and the furnace.

TRAVELING ENGINEERS' ASSOCIATION.—The 12th annual meeting was held at the Auditorium Hotel, Chicago, on Sept. 3 to 6, with a large attendance. The official reports showed a membership of 630, and a balance of about $800. After the opening addresses on the morning of Sept. 3, Mr. W. G. Wallace presented a review of the proceedings of the convention of the American Railway Master Mechanics' Association. This is an annual custom, intended to give the members an idea of what the higher officers of the mechanical department deal with at their meetings. A committee report was then read on how to locate the fault of an engine not steaming properly, without adjusting the draft appliances. The animated discussion turned on the question of whether the smoke-box should or should not be air-tight. Some speakers considered it absolutely essential if the engine is to steam properly, while others thought that even considerable leaks had little effect, but the former opinion was the more general and the more correct.

At the meeting on Sept. 4 the first paper was by Mr. D. C. Buell, on "The Waste of Energy in Railway Operation." This discussed the various losses in the engine itself by the waste of fuel, heat and steam; and then the various causes of waste of time at the roundhouse and on the road. A committee report was then presented on "The Advantage of the Hot-Water System of Washing Out and Filling Boilers." This system as in use on the Pittsburg & Lake Erie Ry. was described in our issue of Nov. 3, 1904, and the report mentioned 27 locomotive terminals thus equipped. The results have been generally satisfactory, but it was remarked by one speaker that cold water would more effectually remove the wind and could be used if there was time to let the boiler cool down. Others thought the hot water had an advantage, and in any case the reduction of the time which engines spend in the roundhouse is one of the special advantages of the hot water system. At the afternoon session there was presented the report of the committee on "The Best Methods of Eliminating the Smoke Nuisance with Locomotives Burning Soft Coal." The conclusions were that coal of one grade should be used as far as possible, and should be properly broken up before being put on the tender; the single-scoop system of firing should be adopted, and more progress made with the application and special use of mechanical devices. In the discussion, mention was made of the improvements effected where city authorities had made an active campaign against the smoke nuisance, and the methods used at Cleveland under these conditions were described.

The first paper read on Sept. 5 was by Mr. J. C. Currie on "The Lubrication of Cylinders and Valves of Locomotives Using Saturated and Superheated Steam." He expressed the opinion that hydrostatic sight-feed lubricators can be successfully employed on engines using superheated steam, and that mechanically operated lubricating devices are not necessary on such engines. One difficulty suggested in the discussion was the long time required for the oil to pass from the sight-feed lubricator to the valve, so that trouble may ensue unless the lubricator is opened some time before the engine starts. A paper by Mr. W. G. Wallace on "The Reduction in Coal Consumed and Increased Efficiency of Men and Locomotives" was devoted to matters of discipline and improvement in the mechanical and transportation departments. Mr. D. R. McBain also read a paper on "The Relation of the Traveling Engineer to Enginemen and the Pooling System." He approved of the pooling system, and his views were at first endorsed, but afterwards this action was revoked as the association last year passed a resolution unfavorable to this system. At the afternon session Mr. Grantier read a paper on "The Advantages of Automatic Stokers as Compared with Hand Firing." He strongly advocated the use of these devices, and stokers of different makes were favorably mentioned in the discussion.

The subject of superheated steam occupied the closing session on Sept. 6, and was introduced by the presentation of a committee report on "Superheated Steam and the Best Method of Getting Good Results when Engines are in Service on Trains." This report described the general features of some of the superheaters in use, particularly in regard to their relation to the work of the enginemen. It was also pointed out that the engineman must do his part in order to make the superheater successful. Officers were elected as follows: President, A. M. Bickel; Secretary, F. O. Thompson, East Buffalo, N. Y. The next convention will be held at Detroit, Mich., in September, 1906.